Methods for Determination of Coastal Development Setback Lines in South Africa.

by
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Declaration

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Abstract

Implementation of the ICM Act (2008) has made it a legal requirement to determine coastal setback lines in all the coastal provinces of South Africa. Coastal development setback lines (or "coastal management lines") need to make provisions for physical coastal/marine processes, as well as "softer" more subjective issues and considerations, e.g. environmental, public access, heritage, sense of place, aesthetics, etc. Both the literature review and recent setback line workshops held in South Africa have highlighted the lack of consistent methods to determine setback lines, as well as the major confusion around how to proceed. The literature review found that the primary coastal processes components of setback lines were related to coastal flooding levels and coastal erosion. Both of these, including sub-components, were not satisfactorily dealt with in terms of methods applied to date. To alleviate these problems, appropriate setback line methods are sought for "data poor" environments, that can be efficiently applied in large study areas, but that are still sufficiently robust and defendable. In view of South Africa's generally very exposed coastline (and the possibility of progressive climate change impacts), the escalating South African coastal development, and the above mentioned problems, the need for appropriate, practical and implementable methodologies to determine setback lines is clear.

This thesis describes the author's research concerning methods for the determination of coastal development setback lines in South Africa, and focuses strongly on the abiotic (geophysical) components of setback lines.

Geophysical coastal hazards and spatial vulnerability, and their relevance to setback lines are considered. A practical coastal hazard and vulnerability evaluation technique, applied to European coastal conditions but applicable to South African conditions including poor data availability, was adapted and further developed (building on methods proposed by Theron *et al*, 2010a, 2012), to include additional forcing factors considered to be relevant under South African conditions.

Following an extensive literature review and testing of several different wave runup models against local data, it is concluded that the models of Nielsen and Hanslow (1991) and Mather *et al* (2011) are the best of the available models and are adequate for application in South Africa, but should be used with certain adaptations as recommended herein. New methods were developed and two alternative approaches are proposed to predict short-term shoreline erosion, requiring less input data, and that are also suitable for larger scale approaches (rather than being limited by the constraints of conventional methods). Current methods of determining setback lines have not adequately taken dune effects into account. Thus a novel approach is proposed for quantifying dune effects on normal shoreline erosion estimates.

Other important components of and requirements for setback lines are dealt with. Thus discussions and specific recommendations, suggestions and guidance are provided on another eight components/aspects necessary for determining setback lines. Finally, all the necessary setback line methodologies and aspects are put together, explaining how they should be applied. The basic components are catalogued and a compilation of the steps required to determine coastal development setback lines is provided. Recommended procedures and methods for conducting/completing each of the steps are given.

Opsomming

Die implementering van die Geïntegreerde Kusbestuurs-wetgewing van 2008 het dit ‘n wetlike vereiste gemaak om kus-ontwikkelings-terugsetlyne te bepaal in al die kusprovinsies van Suid-Afrika. Kus-ontwikkelings-terugsetlyne (ook bekend as kus-bestuurslyne) moet voorsiening maak vir fisiese kus/mariene prosesse, asook “sagter” meer subjektiewe aspekte en oorwegings, byvoorbeeld omgewingsimpakte, publieke toegang, erfenis, estetika/besienswaardigheid, gewaarwording van skoonheid/”plek-sin” (‘sense of place”), ens. ‘n Literatuur oorsig sowel as onlangse werkswinkels wat in **Suid-Afrika** gehou is oor terugsetlyne, het beklemtoon dat daar ‘n gebrek is aan konsekwente metodes/tegnieke om terugsetlyne te bepaal, asook dat daar grootskaalse verwarring heers oor hoe om met die bepaling daarvan voort te gaan. Die literatuur oorsig het bevind dat die primêre kusproses-komponente van terugsetlyne verband hou met vloedhoogtes langs die kus sowel as kuserosie. Beide hierdie komponente asook sub-komponente, was onbevredigend hanteer in die metodes wat tot dusver toegepas is. Om hierdie probleme te oorkom, word toepaslike terugsetlyn metodes benodig, wat geskik is vir beperkte data beskikbaarheid. Dié metodes moet ook effektief toegepas kan word in groot studie-areas, maar moet steeds betroubaar en onaanvegbaar wees. Dit is duidelik dat in die lig van Suid-Afrika se algemeen baie blootgestelde kuslyn (en die moontlikheid van ergerwordende klimaatsveranderings-impakte), die snelgroeiende Suid-Afrikaanse kusontwikkeling, en bogenoemde ander probleme, daar die behoefte bestaan vir toepaslike, praktiese en implementeerbare metodes om terugsetlyne te bepaal.

Hierdie tesis beskryf die outeur se navorsing oor metodes om kus-ontwikkelings-terugsetlyne in Suid-Afrika te bepaal, en is sterk gefokus op die abiotiese (geofisiese) komponente/aspekte van terugsetlyne.

Geofisiese kus-gevare en ruimtelike kwesbaarheid, en hul belang vir/op terugsetlyne word oorweeg. ‘n Praktiese kus-gevaar en kwesbaarheid evaluerings-tegniek, voorheen toegepas op Europese kus-toestande, maar toepaslik vir Suid-Afrikaanse toestande en data beskikbaarheid, is aangepas en verder ontwikkel (gebaseer op metodes voorgestel deur Theron *et al*, 2010a, 2012), om bykomende faktore in te sluit wat toepaslik is vir Suid-Afrikaanse toestande.

Na ‘n uitgebreide literatuur oorsig en die toets van verskeie golf-oploop modelle teen plaaslike data, is bevind dat die modelle van Nielsen en Hanslow (1991) en Mather *et al* (2011) die bestes is van die beskikbare modelle en dat hul voldoende is vir toepassing in Suid-Afrika, maar dat hul gebruik behoort te word met sekere aanpassings soos in hierdie tesis aanbeveel word. Nuwe metodes is ontwikkel en twee alternatiewe benaderings word voorgestel om korttermyn kus-erosie te voorspel, wat minder invoer data benodig, en wat ook geskik is vir toepassings op groter skaal (eerder as om beperk te word deur die tekortkominge van gebruiklike metodes). Huidige metodes om terugsetlyne te bepaal het nie duin-effekte voldoende in ag geneem nie. Daar word dus ‘n nuwe benadering voorgestel om duin-effekte op normale kusereosie beramings te kwantifiseer.

Ander belangrike komponente van, en benodigdhede vir terugsetlyne word ook behandel. Besprekings en spesifieke aanbevelings, voorstelle en riglyne word voorsien wat handel oor ‘n verdere agt komponente/aspekte wat benodig word vir die bepaling van terugsetlyne. Ten slotte word al die benodigde terugsetlyn-medodes en aspekte bymekaar gebring, en word daar verduidelik hoe hul toegepas behoort te word. Al die basiese komponente word gelys en ‘n samestelling van die stappe wat benodig word om kus-ontwikkelings-terugsetlyne te bepaal, word gegee. Aanbevole prosedures en metodes word gegee om elk van die stappe uit te voer en te voltooi.

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Dedications

I would like to dedicate this work to my wife Hannalie, my children Danie and Erica, my Mother and the memory of my Father.

Photograph courtesy of Clark Little

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List of Abbreviations

Abbreviation	Designation or connotation
CD	Chart Datum
CC	Climate Change (Refers to any long-term trend in mean sea level, wave height, wind speed, drift rate, etc.)
CZM	Coastal Zone Management (The development of a strategic, long-term and sustainable land use policy, sometimes also called shoreline management.)
CSIR	Council for Scientific and Industrial Research.
DEA	Department of Environmental Affairs
EIA	Environmental Impact Assessment
HAT	Highest Astronomical Tide (HAT is the highest predicted astronomical tide under average meteorological conditions over a full 18.6-year nodal cycle and is not reached every year.)
HWM	High-Water Mark
ICM	Integrated Coastal Management
KZN	Kwa-Zulu Natal (province located along east coast of South Africa)
LiDAR	Light Detection and Ranging technology (Airborne LiDAR technology can be used to obtain coastal topographical information or data.)
MHWS	Mean High-Water Springs (The average height of the high water occurring at the time of spring tides.)
MSL	Mean Sea Level (The average height of the surface of the sea for all stages of the tide over a 19-year period, usually determined from hourly height readings.)
NCEP	National Centre for Environmental Prediction
SBL	Setback Line
SLR	Sea Level Rise (The long-term trend in mean sea level associated with Global warming.)
TNPA	Transnet National Ports Association of South Africa
WGS	World Geodetic System (1984, revised 2004; An earth-fixed global reference frame used for defining coordinates when surveying and by GPS systems.)

Chapter 1: Introduction

1.1. Background

The importance of sustainable development of South Africa's coastal resources has been recognised by national government as reflected in, e.g. the White Paper on Sustainable Coastal Development in South Africa (DEAT, 2000), the Marine Living Resources Act 1998, and the recent National Environmental Management: Integrated Coastal Management Act (Act No. 24 of 2008) (hereafter the ICM Act) which inter alia calls for coastal management plans. Many of the problems in coastal areas relate to escalating conflicts between development, and environmental protection and management of natural resources (Celliers *et al*, 2009; Mead *et al*, 2013). The promulgation of the ICM Act (2008) means that no new development (including redevelopment of existing developments) will be authorized within 100 m of the high-water mark in South Africa, without first determining an appropriate "coastal development setback line". In fact, setback lines have been determined on an ad hoc basis in South Africa to manage coastal development (albeight with limited scope), long before promulgation of the ICM Act in 2008 and even before the White Paper of 2000, at least since 1990 (e.g. CSIR, 1990, 1991). The ICM Act aims to "achieve sustainable coastal development through a dedicated and integrated management approach". If the current fast tracked development of the South African coast is to occur in a communally beneficial and sustainable manner, it is vital that planning takes place based on scientific knowledge. The objectives of the ICM Act (2008) refer to the need for such research, that it should be on-going and for the co-ordination of the information obtained.

More than 37 % of the global population live within 100 km of the coast (Syvitski *et al*, 2005). Similarly, more than thirty percent of South Africa's population currently lives near the coast, and more than eighty percent of this coastline comprises of sandy shores susceptible to large variability (Tinley, 1985; Jackson and Lipschitz, 1984). It is well known that developers or land owners place a very high premium on being "as close to the sea as possible" and with the best possible "sea views." Locating fixed structures within reach of coastal processes can either interrupt the processes resulting in damage to or loss of property or ongoing maintenance costs resulting from either beach erosion or wind-blown sand inundation or both (e.g. Figure 1.1). The asset which attracts the structure in the first place may be lost or endangered. The high costs required in maintaining developments within the littoral active zone are indicative of a lack of recognition of the prevailing natural processes in the original plans. Quantifying the coastal response to physical environmental factors such as waves, currents, elevated water levels and winds is of prime importance when assessing or planning development initiatives in the coastal zone (Theron, 2004). It is understandable therefore that a holistic insight of coastal processes is becoming increasingly important, not only for the design of coastal developments, but also for the cost effective and environmentally sympathetic management of coastal areas (Dorst and Wilde, 2003). An understanding of the response of a beach to external forces is of great importance in order to define erosion and development setback limits and other possible constraints that must be considered during planning or management actions. Thus, aspects such as wave energy, sand budgets, future sea levels and potential storm erosion setback lines need to be well addressed (Theron, 1994) especially in local studies which focus on the particular coastal types and characteristics of the South African coastal zone.

Figure 1.1: Present South African example of railway line located too close to the sea (Photo: A Theron)

A recent study of the KZN coast (Celliers, 2006 in Mather, 2011) has shown that the strip of land 100m inland of the high water mark has been transformed from being 28% urbanised in 1994 to 50% in 2006 (excluding the Isimangaliso Park, formally called the Greater St Lucia Wetlands Park). In this province, the urbanised coastal area has thus effectively doubled in the last decade. In view of the escalating South African coastal development, our generally very exposed coastline and predictions of accelerated climate change impacts, the need for strong methodology to determine setback lines is clear.

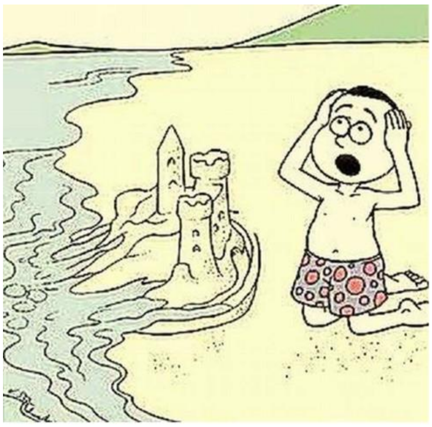
Coastal management plans are receiving unprecedented attention along the coastlines of the world from authorities, planners, developers, and environmentalists. This is due to increasing pressure for development on the coastal zone in the face of natural coastal and environmental processes, increasing storm impacts and (perceived) rising sea levels. In response, South Africa has taken measures to enforce careful planning and allocation of the fragile areas closest to our shores by implementing the Integrated Coastal Management Act (Act No 24 of 2008) in 2009. In particular the ICM Act (2008) legislates the establishment of coastal setback lines in order to protect private and public coastal property, including the natural coastal environment. Underpinning Clause 25 of the ICM Act, coastal setback lines demarcate safe coastal areas, enable the definition of areas that are at risk of being eroded or impacted by coastal processes, and enable the identification of infrastructure that is potentially vulnerable to the effects of sea level rise and inundation due to wave runup. The ICM Act, as well as associated NEMA (National Environmental Management Act) legislation and the Ocean Management Green Paper, indicates principles and pathways to achieving conservation and sustainable development of our coast. In this regard, the Department of Environmental Affairs (DEA) have stated that it is imperative that studies should establish both norms and standards for a uniform approach to the development and demarcation of setback lines for the entire South African coast, as there are several approaches currently being utilised by the coastal provinces and municipalities (DEA Coastal Setback Line meeting of 14 May 2014, chaired by Director Coastal Conservation Strategies Mr L Mudau). (Coastal development setback lines are now also referred to as “coastal management lines” as per the draft ICM Amendment Bill of 2013.)

The determination of appropriate, practical and implementable coastal setback lines will facilitate improved planning and management of coastal areas, as well as result in improved protection being given to the coast. The most vulnerable areas along the coast in terms of abiotic (geophysical) impacts from the sea, will almost invariably be located where problems are already being experienced at present (Figure 1.2). In most cases these are the areas where development has encroached too close to the sea or at too low an elevation above mean sea level (Theron, 2007).

The need for methods of determining setback lines that are robust, practical and implementable on regional (/national) scale has become abundantly clear over the last 5 years (for example, as evidenced in the setback line seminars/workshops held at the University of Stellenbosch in 2010 and 2011). Guidelines for determining consistent and comprehensive setback lines along the South African coast need to be drawn up as a matter of priority in the various provinces, but the methodologies (and their application) have not yet reached a sufficient stage of maturity in South Africa. Therefore, this thesis is also aimed at providing input and contributing towards determining and implementing standard methods of practice for determining coastal setback lines for the entire South African coast.

Figure 1.2: South African examples of existing vulnerable coastal assets due to being located too near the sea (a, c, d and e located in False Bay near Cape Town; b located along KZN coast)

Ultimately it is not nature that is “cruel” to us (as in the cartoon in Figure 1.3), but building our “castles” too near the (indifferent) sea, i.e. lack of appropriate setback, that leads to many of our disasters in the coastal zone.



Oh, cruel nature, WHY?

Figure 1.3: Appropriate setbacks will protect our “castles” (assets, goods & services), from “cruel nature” (the sea) *(Cartoon courtesy of L Celliers)*

1.2. Problem statement

Implementation of the ICM Act (2008) has made it a legal requirement to determine coastal setback lines in all the South African coastal provinces. However, to date, these setback lines have been determined in an ad hoc manner by practitioners on behalf of various municipal and provincial authorities in an inconsistent manner, with most of the South African coast still not yet covered. Both the literature review and recent setback line workshops held in South Africa have highlighted the lack of consistent methods to determine setback lines as well as the major confusion around how to proceed. In addition, the various technical methods that have been applied to determine setback lines in South Africa, to date mostly have specific shortcomings. Adequate methods do exist to conduct fine scale, detailed setback studies in small coastal study areas. However, these “detailed” methods rely on comprehensive input data, which is largely not available in South Africa and which means that the models employed cannot usually be calibrated or verified for the study area. These “detailed” methods are also time consuming, require data that is expensive to acquire, and cannot practically be “rolled out” to large study areas. To alleviate these problems, appropriate setback line methods are sought for “data poor” environments, that can be efficiently applied in large study areas, but that are still sufficiently robust and defendable.

In view of South Africa's generally very exposed coastline (and the possibility of progressive climate change impacts), the escalating South African coastal development, and the above mentioned problems, the need for appropriate, practical and implementable methodologies (and associated guidelines) to determine setback lines is clear. *The "problem statement / research question" can be expressed relatively concisely as: "how best can appropriate coastal development setback lines be determined for "data poor" South African environments, that can be efficiently applied in large study areas, but that are still sufficiently robust and defendable?"* Although adequate methods do exist to quatify aspects such as coastal erosion and flooding in fine scale, detailed studies in small coastal study areas, there are gaps in terms of methods that can be applied at regional or larger scale and that are realistic despite a paucity of input data. One of the aims of this research is therefore to find or develop methods that address these science gaps, which are still sufficiently robust and applicable to the various South African coastal environments.

Thus, the research objectives are to:

- Develop remedies for specific technical shortcomings in the methods that have been applied to determine setback lines in South Africa;
- Find or derive appropriate setback line methods for "data poor" environments, that can be efficiently applied in large study areas, but that are still sufficiently robust and defendable;
- Make recommendations (and provide associated guidance) for appropriate, practical and implementable methodologies to determine setback lines in South Africa.

1.3. Delineations and limitations

The focus of this thesis is strongly on the abiotic (geophysical) components of setback lines, but environmental and social aspects of setback lines will also be briefly discussed. Although the application of setback lines has a strong legal connotation (according to the South African ICM Act, 2008), the focus of this study is similarly not on addressing the legal aspects.

1.4. Definition of key terms

Key terms used in this thesis are briefly defined below, while more thorough descriptions are provided in the relevant thesis sections. A comprehensive glossary of terms in included in Appendix 1.

Setback

Historically commonly used in CZM and coastal engineering terms as a required distance landward of a selected contour line (or the shoreline) to safeguard, for example, infrastructure from marine impacts (such as storm waves or erosion). Currently the definition has been broadened to include protection or conservation of natural areas and additional socio-economic assets in the coastal zone.

Erosion setback line

The *erosion setback line* is a line indicative of the expected landward limit of erosion of a beach or coastline (due to for example sea storms or long-term recession of the shoreline). Areas within the dynamic littoral active zone can erode (e.g. Figure 1.4), as well as accrete.

Figure 1.4: Wave erosion of Southern Cape shore in progress *(Photo A Theron, 2008)*

The erosion setback (or recession) line is usually located at the most landward extremity that the location of a chosen contour line is expected to exceed only once within a specified time-period (Figure 1.5). The acceptable risk is traditionally considered to be that the location of the erosion setback line should not be exceeded more than once in 50 years.

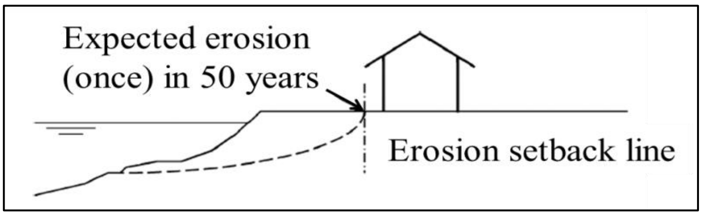


Figure 1.5: Concept of an erosion setback line.

Coastal processes setback line

The *coastal processes setback line* is a line indicative of the expected landward limit of damage or impact at the coastline, due mainly to erosion or wave impacts or flooding from the sea, but also includes (part of) the littoral active zone, which extends to the expected landward limit of coastal sediment transport (through wind and wave action). The coastal processes setback line, therefore, mainly deals with the geophysical coastal-marine processes and dynamics. Additional management areas that may be required to deal with the geophysical coastal processes such as foredune areas and vegetated buffer areas (by means of which wind-blown sand problems can be managed), are therefore also included in this definition. The coastal processes setback line can also be defined as the line landward of which fixed structures may be erected with reasonable safety against the physical impact of the abiotic coastal processes (these typically being winds, waves, currents, sediment transport, etc.). (Fixed structures include buildings, ablution facilities, parking areas, roads, etc.)

Coastal development setback line

The *coastal development setback line* is a line indicative of the position from which all development (e.g. infrastructure, amenities, housing) *should* ideally be located landward. The area seaward of this line incorporates the coastal processes line, as well as an area mainly required to manage usage and conservation of the coast. Besides the coastal processes, coastal development setback lines also have to consider other aspects, mainly related to ecological and social issues (as discussed in Chapter 8). In South Africa, coastal development setback lines are in fact now also referred to as “coastal management lines” as per the draft ICM Amendment Bill (2013).

Coastal flooding level or elevation

In this thesis *coastal flooding elevation (or level)* is defined as the highest point that the seawater can reach at the shoreline, due to the effects of natural events such as tides, winds and storm waves, which may be exacerbated in the long-term by processes such as sea level rise. The implication in not that areas located within the coastal zone below this level would permanently (or for extended periods of several days or longer) be inundated (“flooded”) by seawater. During extreme events such as sea storms (resulting in surge and/or wave runup), thus for relatively short periods (ranging in the order from typically seconds to hours), the seawater may reach up to a certain elevation on the shoreline, which is here called the coastal flooding level or elevation.

1.5. Brief chapter overview

Chapter 1 gives the background to coastal setback lines, and discusses the rationale for this thesis and the objectives of this research. A definition of key terms related to setback lines is provided and delineations and limitations of this thesis are also discussed. The chapter closes with this brief chapter overview.

Chapter 2 contains a literature review of both the international and South African practice or methods of determining setback lines, as well as detailed discussions of specific articles or documents (authors or aspects) pertinent to components thereof. Based on these reviews and findings, conclusions are reached which inform the direction and specific aspects of this study. Having thus identified which specific aspects of setback lines needed to be addressed, these were studied in detail in the succeeding chapters.

Chapter 3 gives a brief overview of how the study was approached and what research methods were followed, as well as which types of data was utilized.

Geophysical coastal hazards and spatial vulnerability, and their relevance to setback lines are considered in Chapter 4.

Extreme seawater levels, wave runup prediction and coastal flooding elevations in South Africa are collectively one of the major components of setback lines and are analyzed in Chapter 5.

Shoreline changes and coastal erosion is the subject of Chapter 6, leading to new methods related to predictions of erosion and determination of long-term trends, which are also major components of coastal setback lines.

A practical method to account for the additional shoreline erosion protection provided by dunes in determining setback lines in South Africa is developed in Chapter 7, which is a novel approach to address this previously unaccounted for aspect of setback lines.

Other important components of and requirements for setback lines are dealt with in Chapter 8. Thus discussions and specific recommendations, suggestions and guidance are provided on additional components or aspects necessary for determining setback lines.

Based on the foregoing, the required components of coastal development setback lines are given in Chapter 9, as well as how these should be determined. Recommendations on both the methods for and the actual determination of setback lines in South Africa, as well as guidance to assist in the application of such methods are provided.

A summary of findings and conclusions is presented in Chapter 10, as well as recommendations for future research.

The thesis ends with the list of References.

Chapter 2: South African coastal characteristics and review of setback line methodologies

2.1. Introduction

This chapter first provides a systematic characterisation of the various South African coastal environments. A review and discussion of South African and international literature concerning setback line methodologies as well as aspects directly related to determining setback lines is then provided. It includes a discussion or evaluation of historical and current approaches followed in South Africa. The aim is to inform both the direction and specific aspects of this study, as well as to identify which particular facets of setback lines need to be addressed or studied in detail. Together, the background to setback line methods (Section 1.1), the problem statement (Section 1.2), and the literature review and discussions provided in this chapter, should steer the research conducted for this thesis by addressing these questions:

- What are the characteristics of the various South African coastal regions?
- What are the requirements for coastal setback line methods?
- Which components must be included in determining setback lines?
- What are the shortcomings of current methods?
- Which aspects of setback lines need to be studied?

2.2. South African coastal characteristics

2.2.1 General, oceanographic setting and ocean currents

The South African coastline stretches for approximately 3 100 km from the Orange River mouth on the desert west coast, around the Cape of Good Hope to tropical Ponta do Ouro at the Mozambique border on the east coast (Figure 2.1). The character of this coast is determined by a number of factors, including the geomorphology of coastal regions; the influence of three major marine bodies off the southern African continent – the Indian, Atlantic and Southern oceans; the air-sea interaction generated by these bodies, which has a decisive influence upon the climate of southern Africa; the land-sea interactions, which shapes beaches dunes and estuaries; the impacts of a multitude of human activities which have transformed significant areas of the coast through development and exploitation of resources (Swart *el al*, 1996). The coastline is rugged and exposed, with few natural bays, and consists of long stretches of sandy beaches interspersed by rocky sectors. The typical littoral drift directions are ''up'' the coast from Cape Town along both seaboards, as driven by the highly energetic south westerly swells arriving from 1000s of kilometres away in the Southern Ocean. Nett drift rates vary with local conditions and coastal alignment, but can attain in the order of 1 million m^3 per year (Swart *el al*, 1996). South Africa's coastline spans three bio-geographical regions (or coastal climatic zones), namely the cool temperate west coast which faces the Atlantic Ocean, the warm temperate south coast, and the subtropical east coast which faces the Indian Ocean (Brown and Jarman, 1978). The Benguela Current on the west coast (Figure 2.1) comprises a general equator-ward flow of cold water in the South Atlantic gyre and dynamic wind-driven upwelling close inshore at certain active upwelling sites (Shannon, 1985). The Agulhas Current flows strongly south-eastward along the east coast (Figure 2.1). These ocean currents do however not flow close to the shore, they tend to follow the edge of the continental shelf, which can be represented by the 200 m depth contour.

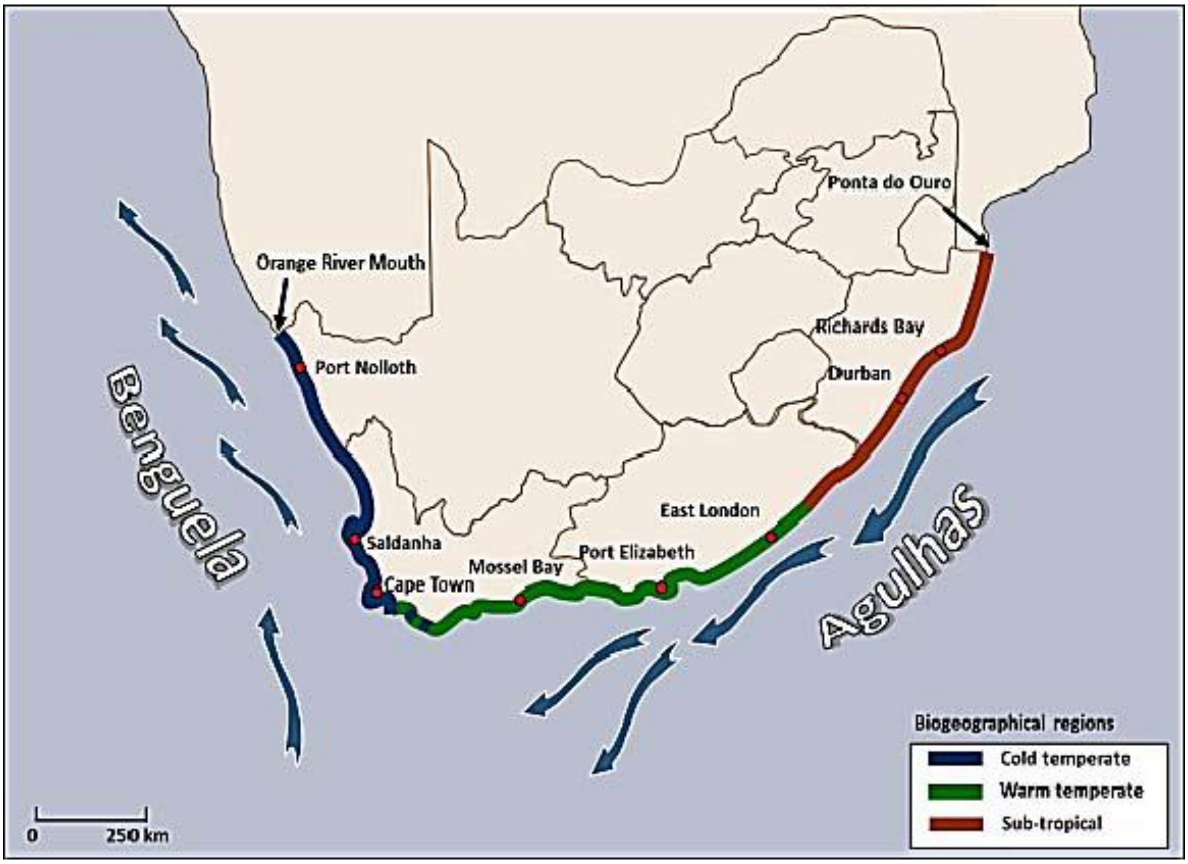


Figure 2.1: Biogeographical regions and currents along the South African coast (Department of Environmental Affairs, 2014).

2.2.2 Bathymetry

The continental margin of the east coast is characterised by an extremely narrow shelf (Figure 2.2). In some places it is only 3 km wide, but in others, it reaches 40 km. The Natal Bight which comprises some 25% of the coastal shelf area adjacent to the path of the Agulhas Current, is a unique feature where the shelf is relatively broader (50km) with a gentler slope than elsewhere along the KZN coast. From Richards Bay to Durban the shelf break occurs at a depth of 100 m; south of Port St. Johns it gradually increases in depth, reaching about 140 m southeast of Cape Recife which is near Port Elizabeth (Flemming, 1981). From Port Elizabeth westward around the southern tip of Africa and up the South African west coast, the shelf is significantly broader with a generally gentler near- to offshore slope.

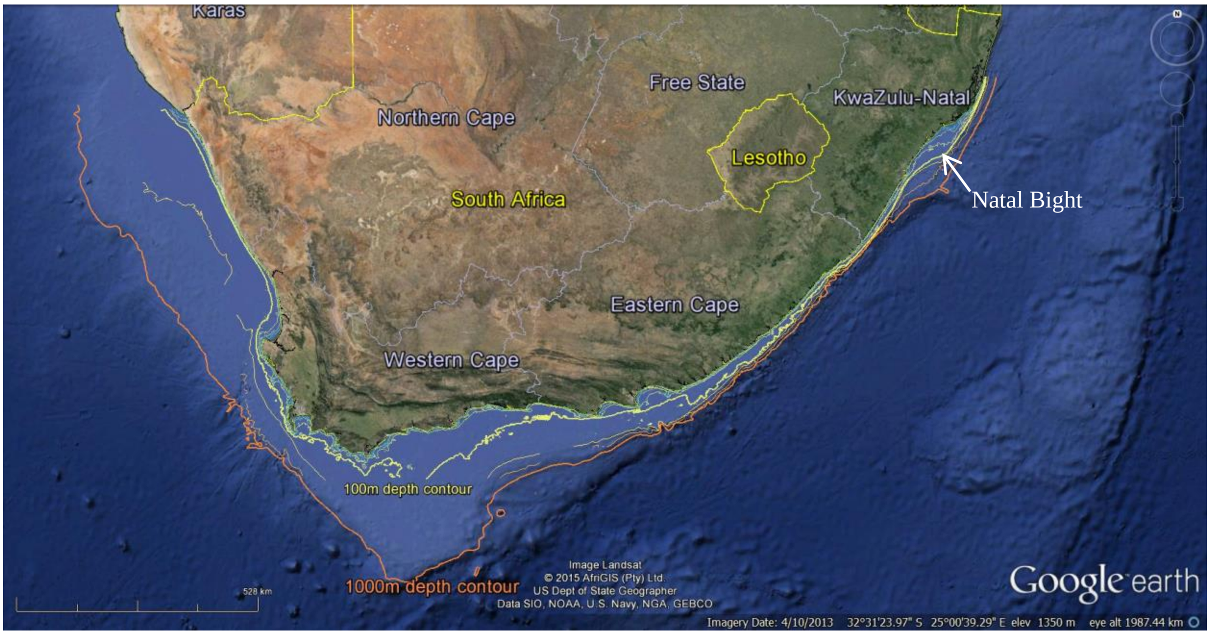


Figure 2.2: Bathymetry along the South African coast.

A GIS analysis of the inshore bathymetry has been undertaken of a number of regions along the South African coast (Rossouw *et al*, 2014). Based mainly on data provided by the South African Naval Hydrographic Office, more than 20 areas along the coast were covered, stretching from Port Nolloth on the west coast to St Lucia on the east coast (the coastal centre-point of each area is indicated in Figure 2.3). Each location covered an approximate area of 100 km along the shoreline; thus the inshore bathymetry was investigated of about 2/3 of the South African coast. By means of the GIS procedure the cross-shore distance from points located at 500 m intervals along the shoreline (along the 0 m MSL contour) to the 15 m depth contour were determined (Rossouw *et al*, 2014).

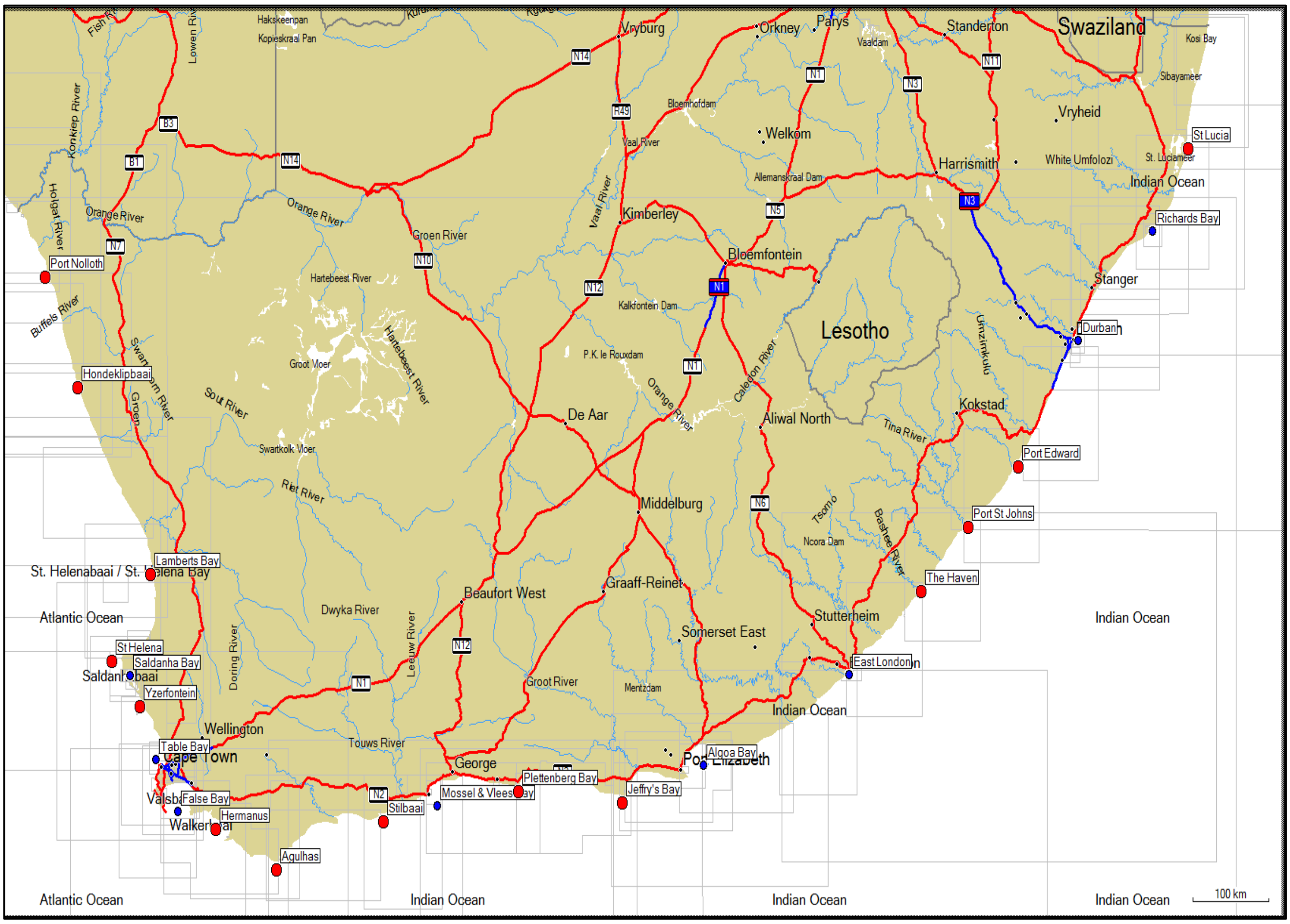


Figure 2.3: Numerical model location map indicating the approximate centre point of each numerical model grid (with red and blue dots; from Rossouw et al, 2014).

As part of this PhD study, these distances were used to calculate the inshore slope (0 to 15 m depth) at each coastal point (at 500 m intervals) within each of the 20 coastal areas of 100 km each. The inshore slope is one of the parameters indicative of the amount of incident wave energy reaching the shoreline, in that mild slopes generally mean a wider zone over which more incident wave energy is dissipated than over steep slopes where relatively more incident wave energy reaches the shoreline (Mather *et al*, 2011). (The inshore slope is also one of the parameters directly employed in some of the methods described in Chapters 5 and 6 to determine wave runup and coastal erosion.) The calculated slope results were analysed statistically to determine slope parameters such as: minimum (i.e. mildest slope), 10 % exceedance, mean (average slope), 90 % exceedance, and maximum (steepest slope) for each of the 20 coastal areas of 100 km alongshore (i.e. ≥ 200 data points per area). The mean inshore slope of all the coastal points (i.e. totalling ≥ 4000 data points over the 20 areas) is 0.0176 (i.e. about 1 in 57), while some 80% of all the points had slopes between 0.0371 and 0.0104 (i.e. between about 1 in 27 to 1 in 96). (It should be noted that coastal points located inside harbours, on breakwaters, and in coastal lagoons and estuaries were eliminated from the analyses.) The analysis was also conducted to ascertain whether there are any regional inshore bathymetric characteristics or patterns that emerge. The results for the 20 areas are graphically illustrated in Figure 2.4, in which the 20 areas are listed in geographical order from east to west around the South African coast (i.e. from St Lucia on the northeast coast around the Cape to Port Nolloth on the northwest coast). The 20 areas all include relatively wide ranges of inshore slopes and no regional inshore bathymetric characteristics or clusterings are observed. Naturally, the steepest inshore slopes are located at headlands and capes, while the mildest slopes occur within bays. This statement goes hand in hand with the further observation that the steepest slopes within each area are all found along rocky shores (typically exhibiting steep gradients on the landward side as well). The milder slopes are mostly found along sandy shores usually within bays or along pocket beaches, while a few mixed sandy/rocky shores also have mild slopes due to reefs located in the surf zone or seaward thereof.

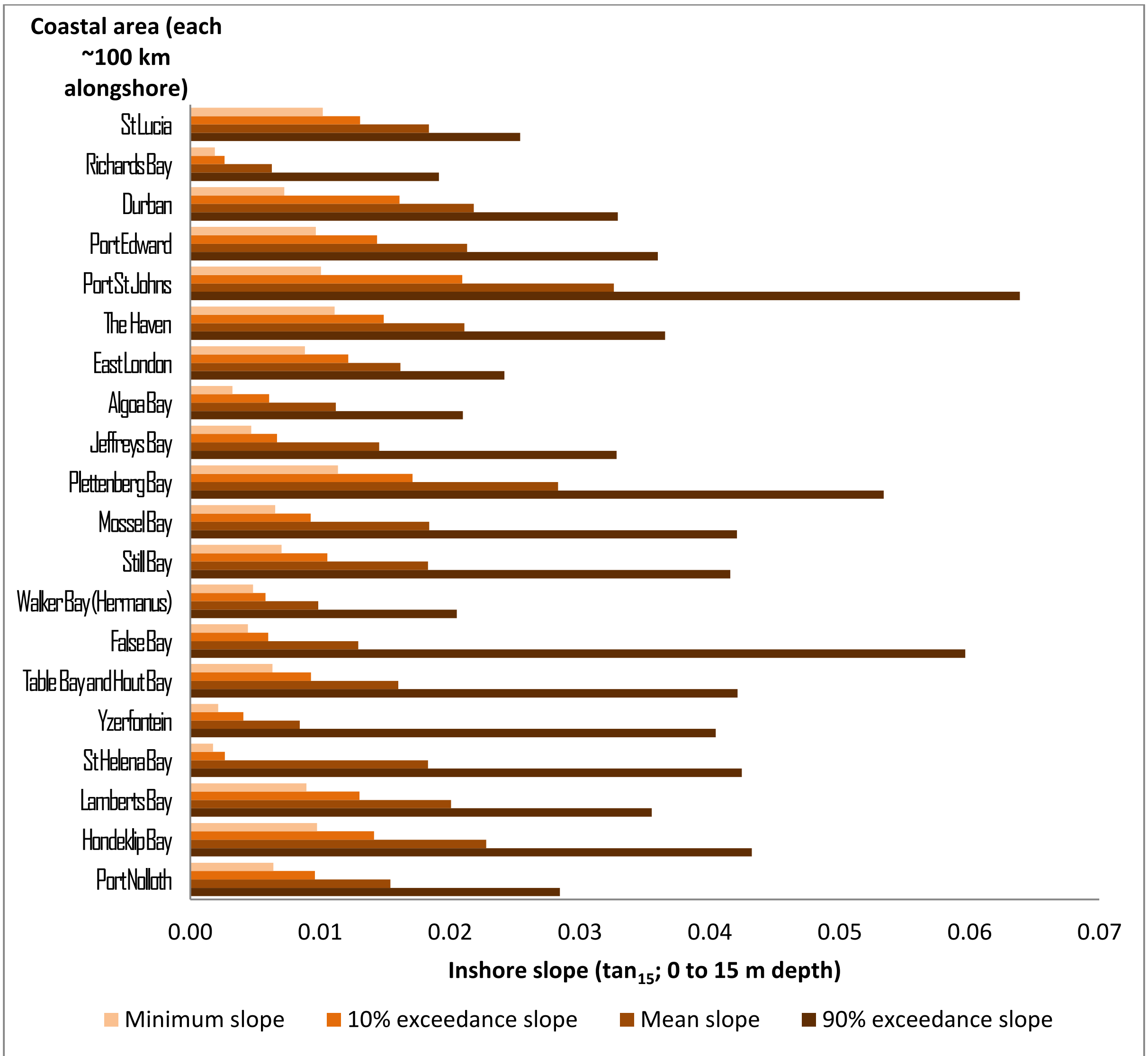


Figure 2.4: Analysis of the inshore bathymetry along the South African coast (from east to west around the coast).

2.2.3 Wave regime

South African offshore wave climate

To understand the South African wave climate it is important to describe the characteristics of the various weather systems off the coast of South Africa that generate waves and cause them to propagate towards the shores. The general weather climate of the southern African oceans is influenced by different types of synoptic patterns (MacHutchon, 2006). A brief overview of the four main types of weather system patterns and resulting wave regimes generated along the South African coast is provided below (adapted from MacHutchon, 2006):

The semi-permanent subtropical high-pressure cells off the west and east coasts (Figure 2.5a). This weather system is responsible for the higher-frequency wave conditions on the west coast. During the summer season, this system generates waves on the west coast that propagate in a north-north-easterly to northerly direction with peak wave periods (T_p) ranging from about 5 to 10 seconds (s).

The cold front system comprising a low-pressure cell associated with a front of cold air coming from the south or south-west (Figure 2.5b). This type of weather system is responsible for most of the wave conditions along the South African coast, which include long-period swell to local sea conditions. The waves can approach from a westerly direction on the west coast to a south-westerly direction on the south coast. The peak period and range vary from 5 s to 20 s. Significant wave heights of more than 10 m can be expected during extreme storm events. During winter, the wave height increases along the coasts as the frontal systems travel along a more northerly trajectory. During summer, the high-pressure systems along the west coast force the low-pressure systems farther south, resulting in a general decrease in wave height along the coast (Rossouw 1989).

Cut-off low (COL) systems (Figure 2.5c). These systems normally consist of a low-pressure cell blocked by two high-pressure cells on either side. COL systems are relatively common but during rare stationary periods can result in extreme storms along the south-east and east coasts.

Tropical cyclones (Figure 2.5d). These systems generally originate in the Indian Ocean, east of Madagascar, generating waves occurring mainly on the east coast, along the Mozambican and South African north-east coast. However, up to the present, only one extreme cyclone wave event has been recorded in South Africa (Rossouw and Theron, 2009).

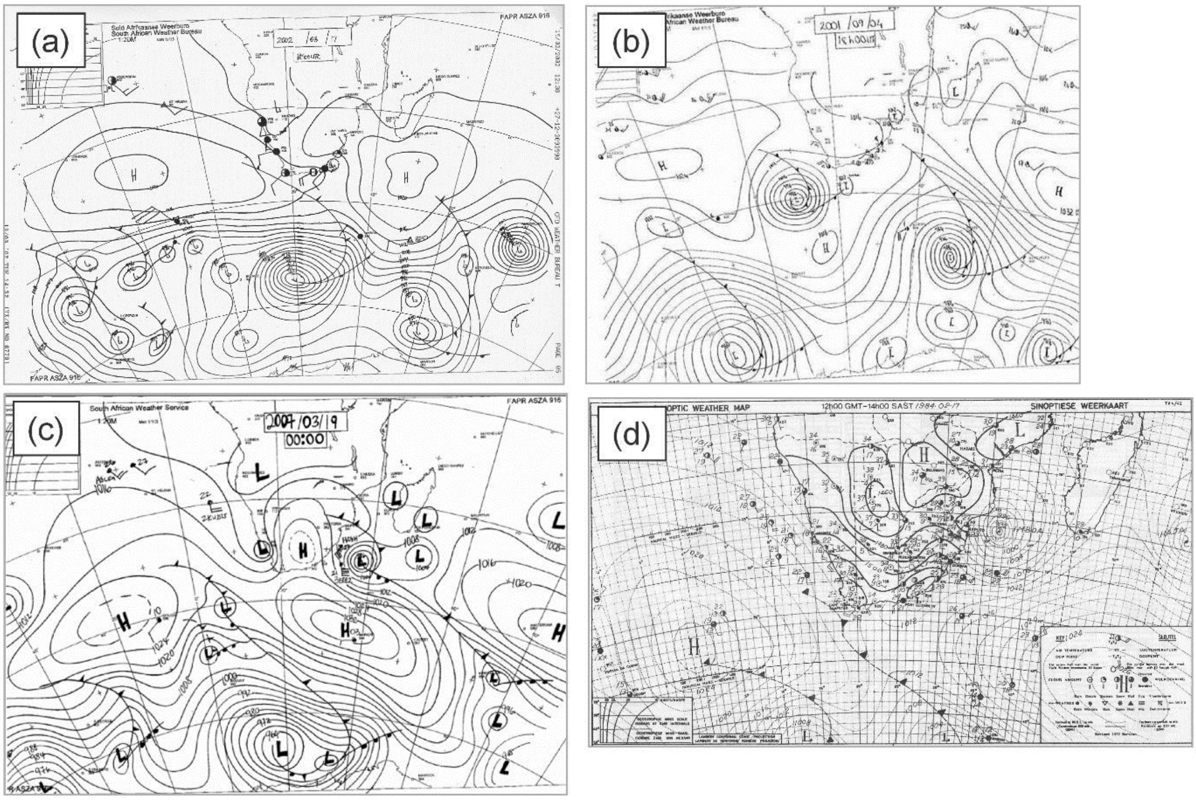


Figure 2.5: Synoptic charts illustrating four types of weather systems mainly associated with the wave climate around the SA coast *(Produced by the South African Weather Service, 1984, 2001, 2002 and 2007)*

A description of the regional wave climate off the South African coast, in terms of general and extreme climates as derived by Rossouw and Theron (2009) is provided here. An overview of the annual variation in wave height and period along the South African coast is given in Figure 2.6. Two wave heights are presented. The first value represents the median significant wave height ($H_{mo}^{50\%}$), which is exceeded for 50% of the time. The second wave height is exceeded for only 1% of the time ($H_{mo}^{1\%}$), giving an indication of the more extreme condition. Also presented is the most likely range of peak wave periods that can be expected for each location. These values are based on about 11 years of WaveWatch III forecast model data (Tolman *et al*, 2002) of the NCEP (NCEP, 2013) since very little measured data were available in the offshore domain.

As indicated in Figure 2.6, the largest waves occur along the south-west towards the south coast but decrease in magnitude moving northwards up both the west and east coasts. This is consistent with information indicating that swell intensity decreases toward the lower latitudes along both sides of South Africa as provided by Flemming (1981) and Rossouw & Rossouw (1999). The distribution of wave period remains fairly constant, due to the swell propagating northwards. In general, peak wave periods range from 4 s, which represent local sea conditions, to about 18 s, which represent long-period swell conditions (Barwell *et al*, 2014). However, all the data indicate that periods of 10 s to 12 s occur most of the time. Thus, the wave climate along the South African coast is swell dominated.

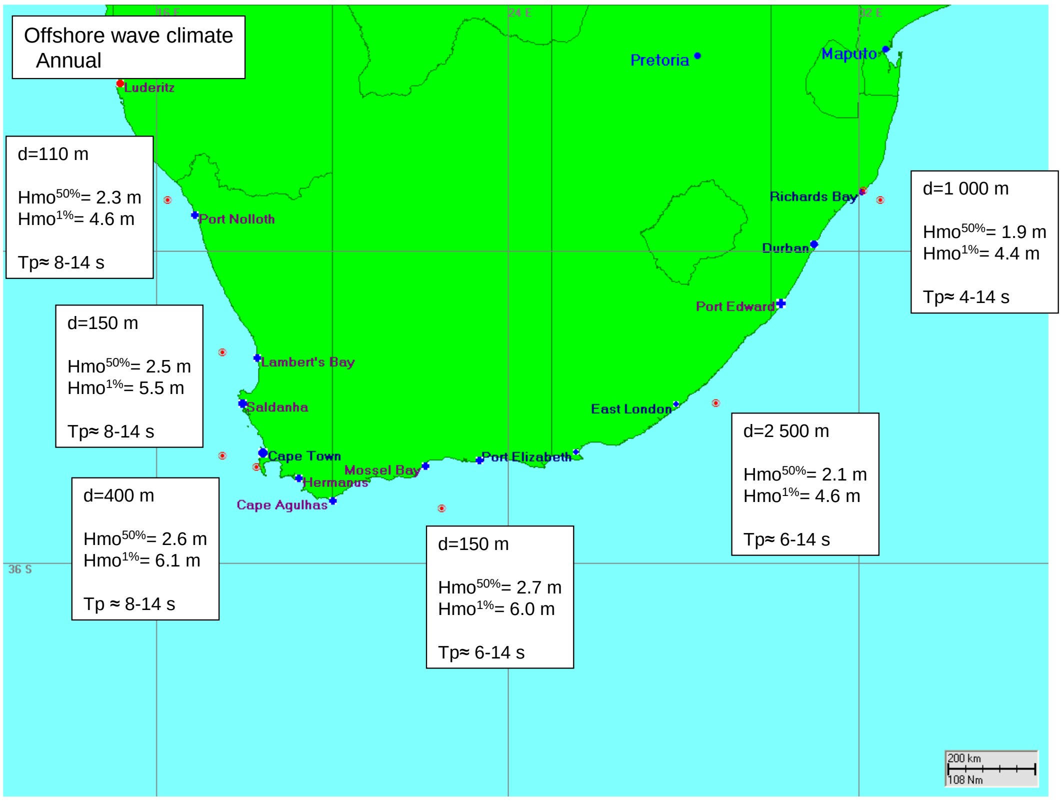


Figure 2.6: Overview of wave height and period distribution around the South African coast.

The annual variation in directionality around the coast is illustrated by the wave roses in Figure 2.7. The dominant wave direction is south-west, representing the general direction of the passing low-pressure systems. Note that during winter months, when the trajectories of the lows are farther northwards, the waves will approach from a more west-south-westerly direction on the Cape south-west coast. Farther northwards on the west coast, there is also a more south-south-westerly component, representing seas generated by the more local southerly wind conditions. Along the Cape south coast the wave direction is still predominantly south-west but swings more toward a south-south-westerly direction on the east coast. Farther northwards on the east coast, there is also a smaller more easterly component, representing seas generated by the more local easterly wind conditions (as well as rare contributions from tropical cyclones in the Mozambican channel).

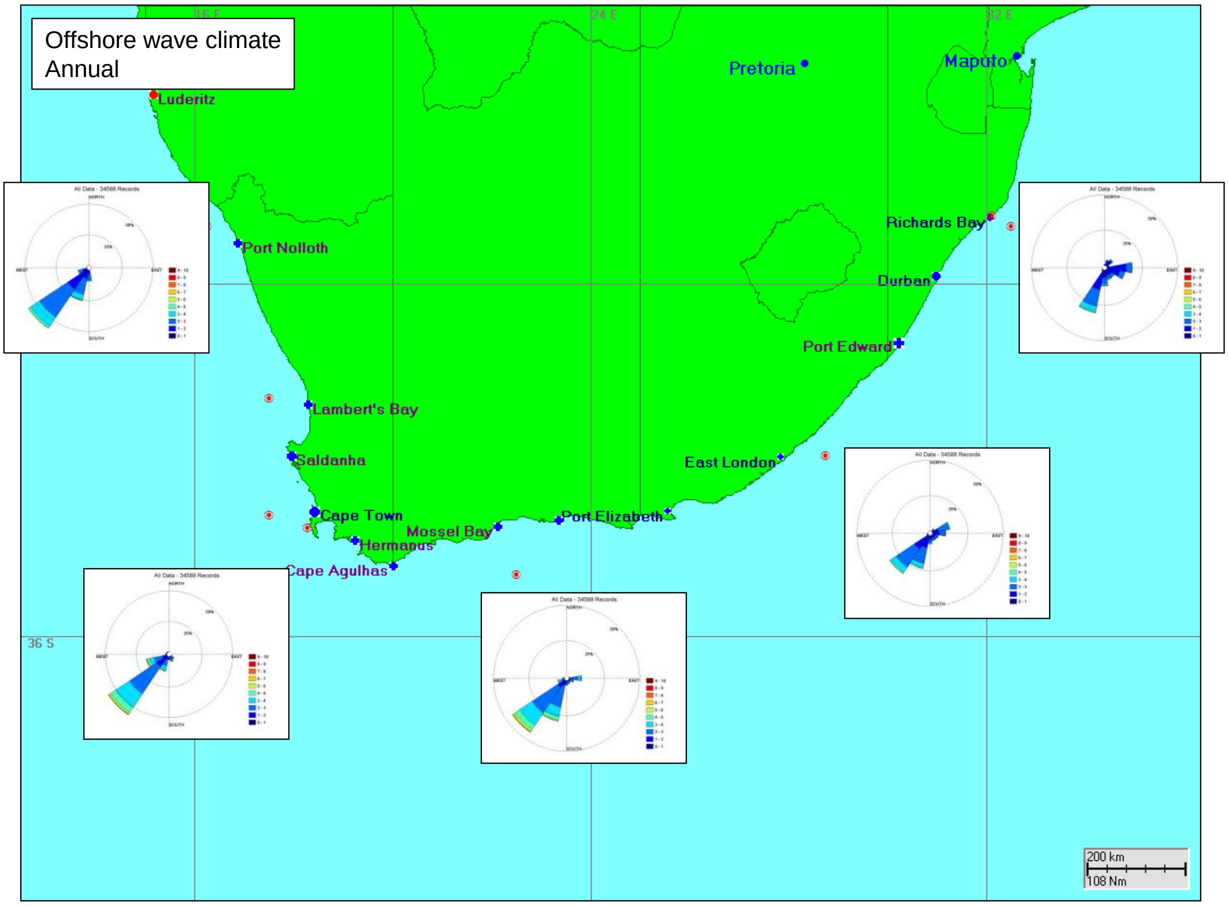


Figure 2.7: Overview of wave directionality around the South African coast

Regional inshore wave climate

To determine the inshore wave climate along the South African coast, Rossouw *et al* (2014) conducted hydrodynamic wave modelling (SWAN, Booij, *et al* 1999), to transform offshore wave data to inshore conditions. The present South African offshore wave climate at deep sea locations around the South African coast was determined by using NCEP hind-cast wave data (NCEP 2013), from the NOAA/NCEP WAVEWATCH III Global Model (Tolman *et al* 2002). A medium resolution wave analysis (0.5 km numerical grid intervals) was undertaken, by setting up numerical wave models for a number of regions along the South African coast (Rossouw *et al*, 2014). In total, more than 20 numerical models were set up for the coast, covering the area from Port Nolloth on the west coast to St Lucia on the east coast. These are the same 20 areas that were covered by the inshore bathymetry analyses (the coastal centre-point of each as indicated before in Figure 2.3). Each model covered an approximate area of 100 km along the shoreline. The locations cover the major municipal regions and coastal towns as well as selected rural and undeveloped natural areas. These numerical models allowed the derivation of the nearshore wave climates for locations at about 500 m intervals approximately along the 15 m isobath in each modelled area. A "medium resolution" inshore wave climate was thus determined for about 2/3 of the South African coast (Rossouw *et al*, 2014). The derivation process basically entailed the transformation of about 15 years of offshore data (the NCEP data) to the nearshore locations (approximately along the 15 m isobath) using the numerical model for each location. Comparisons with measured wave data indicated that the nearshore wave climates were well represented by the modelled data (Rossouw *et al*, 2014). Based on the modelled nearshore data, extreme wave heights were also estimated using a statistical extreme value analysis procedure (Rossouw *et al*, 2015). By means of this procedure the 1-in-10 year significant wave heights were determined for points located at 500 m intervals along the 15 m depth contour covering an approximate area of 100 km along the shoreline for each of the 20 coastal areas around the South African coast.

As part of this PhD study, these extreme significant wave heights were analysed statistically to determine parameters such as: minimum (i.e. lowest extreme wave height), 10 % exceedance, mean (average extreme wave height), 90 % exceedance, and maximum (i.e. highest extreme wave height) for each of the 20 coastal areas of 100 km alongshore (i.e. ≥ 200 data points per area). The inshore wave climate, especially the extreme events is one of the most important drivers of extreme inshore seawater levels and coastal erosion along the South African coast (e.g. Smith *et al*, 2010; Theron *et al*, 2010a). The mean 1-in-10 year significant wave height of all the inshore points (i.e. totalling ≥ 4000 data points over the 20 areas) is 5.6 m, while some 80% of all the points had 1-in-10 year significant wave heights between 4.7 m and 6.5 m. (It should be noted that inshore points located inside breakwaters and coastal lagoons were eliminated from the analyses.) The analysis was also conducted to ascertain whether there are any regional inshore wave climate characteristics or patterns that emerge. The results for the 20 areas are graphically illustrated in Figure 2.8, in which the 20 areas are again listed in geographical order from east to west around the South African coast (i.e. from St Lucia on the northeast coast around the Cape to Port Nolloth on the northwest coast).

In terms of the highest extreme wave heights (1-in-10 year significant wave heights), a clear pattern emerges. The highest inshore wave heights occur along the most southerly located areas (i.e. exposed locations in the Algoa Bay to St Helena Bay areas), with gradually decreasing maximum wave heights moving in a northerly direction up both the east and west coasts. This is consistent with the general northward decay of extreme offshore wave heights off the South African coast (along both the Indian and Atlantic seaboards) as described before and also reported in Rossouw and Rossouw (1999).

If the maximum variation in extreme inshore wave heights (e.g. minimum 1-in-10 year significant wave heights over maximum wave height) within each 100 km coastal area is studied, then further patterns can be distinguished. The east coast from the East London area northward all the way to St Lucia, as well as the west coast from the Lamberts Bay area northward to Port Nolloth both exhibit significantly less variation in extreme inshore wave heights than along the rest of the SA coast. Within these two regions the wave height reductions from the most exposed locations to the most sheltered locations (all along the 15 m isobath) are all in the order of 40% or less. This can be directly ascribed to the characteristics of the coastline configuration in the two regions, in that both are generally linear open coastlines lacking major capes or headlands, and having only relatively few small bays or coves of limited indentation, therefore the smaller variation in alongshore wave heights.

Not surprisingly, the other regions, which are the western Cape, southern Cape and southern portion of the eastern Cape, within which large bays are found, naturally exhibit large alongshore variations in extreme inshore wave heights. The largest wave height reductions from the most exposed locations to the most sheltered locations (all along the 15 m isobath) are found in the False Bay (76% reduction), Table Bay (74% reduction) and St Helena Bay (63% reduction) areas. The bays found in the Southern Cape (e.g. Walker Bay, Still Bay, Mossel Bay) and southern portion of the eastern Cape (e.g. Plettenburg Bay, Jeffreys Bay, Algoa Bay) are less extensive (shallower indentations) and these regions exhibit wave height reductions of up to 59%.

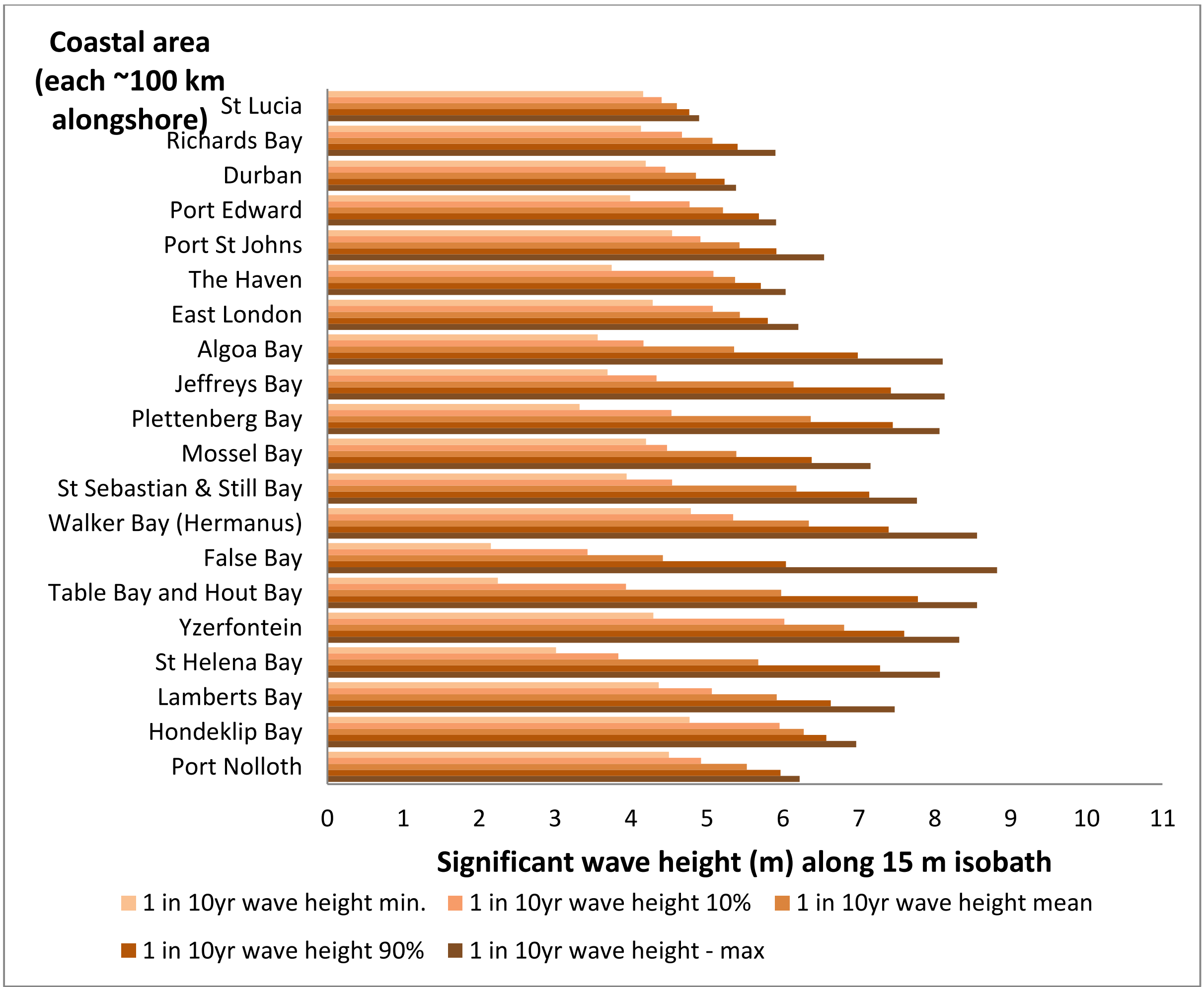


Figure 2.8: Analysis of the inshore wave climate along the South African coast (from east to west around the coast).

2.2.4 Regional geo-physical shoreline characteristics

Coastlines are naturally irregular and the degree of irregularity can be measured by the "fractal index" (Mackie, 1993). The South African coast has an exceptionally low fractal index. It can be described as one of the straightest coasts in the world. As a whole, it has amongst the least amount of embayments, headlands, and lagoons of any coast anywhere (Mackie, 1993). On the other hand, this is a high-energy coast which, coupled to the very low fractal index produces some of the most exposed coastal stretches in the world. Variations in the geo-physical shoreline characteristics are mainly related to the terrestrial geology, climate, tide and wave climate (Cox, 2012). The South African coastal climatic zones and regional wave climate have been described in the previous sections (Sections 2.2.1 and 2.2.4 repectively). As will be discussed in detail in Section 5.2.1, the tides around the South African coast are remarkably uniform, being strongly semi-diurnal in character and having a tidal range of about 2 m. Thus, the tides are in general not a contributing factor in distinguishing between the various South African coastal regions. Due to the nature of the tides and the South African coast, tidal currents are generally relatively low along virtually the entire South African coast, except where tidal flows are locally constricted, for example, at estuary and logoon mouths, or in entrances to ports and harbours.

Shoreline types are generally classified as sandy, rocky or muddy (or combination of these). The coastal zone of South Africa comprises various types of benthic substrate including several sandy, rocky and mixed substrata (Sink *et al.*, 2012). As such, the South African coast contains no muddy shorelines, even inside the most sheltered coastal embayments. The only “coastal” areas where muddy shores are occasionaly found, are those located inside some estuaries and lagoons, which is also where the only mangroves or wetland areas can be found. (Mangroves are only found in some estuaries on the warmer South African east coast). Barrier island coasts are well known in some areas, such as for example in parts of the USA, but also do not occur along the South African coast. Delta coasts is another well known shoreline type that, although found in southern Africa (e.g. Mozambique), does not occur along the South African coast. The absence of ebb-tide deltas at South African river mouths can be mainly ascribed to our high energy wave dominated coast, which prevents deposition of finer sediments on the shoreline (or in the surf zone). Along the sandy coasts, the beach profiles and shoreline configurations are the result of interaction between the prevailing waves (and usually to a lesser degree currents and winds), shoreline orientation, sediment characteristics and sediment sources/sinks.

Regional storm surge levels are determined in Section 5.2.5 and are discussed in detail in Chapter 5.

A classification of the South African coast was conducted by Jackson and Lipschitz (1984), but is considered to be unsuitable for the purposes of this research, as it focussed on coastal sensitivity to oil spills. A brief description of the morphological characteristics of the South African coast by the author is therefore provided here. For the present study the coastline has been sub-divided into five regions (Figure 2.9), on the basis of their morphological characteristics, general orientation and exposure to waves:

(1) North West Coast - Oranjemund (on the SA/Namibian border) to Lamberts Bay (c.a. 200 km north of Cape Town
(2) South West Coast - Lamberts Bay to Cape Agulhas (southernmost point of Africa)
(3) South Coast - Cape Agulhas to Cape Padrone (c.a. 65 km east of Port Elizabeth)
(4) East Coast - Cape Padrone to Mtunzini (c.a. 35 km SW of Richards Bay)
(5) North East Coast - Mtunzini to Ponta do Ouro (on the SA/Mozambican border)

These five regions are illustrated on the map shown in Figure 2.9, and are discussed thereafter.

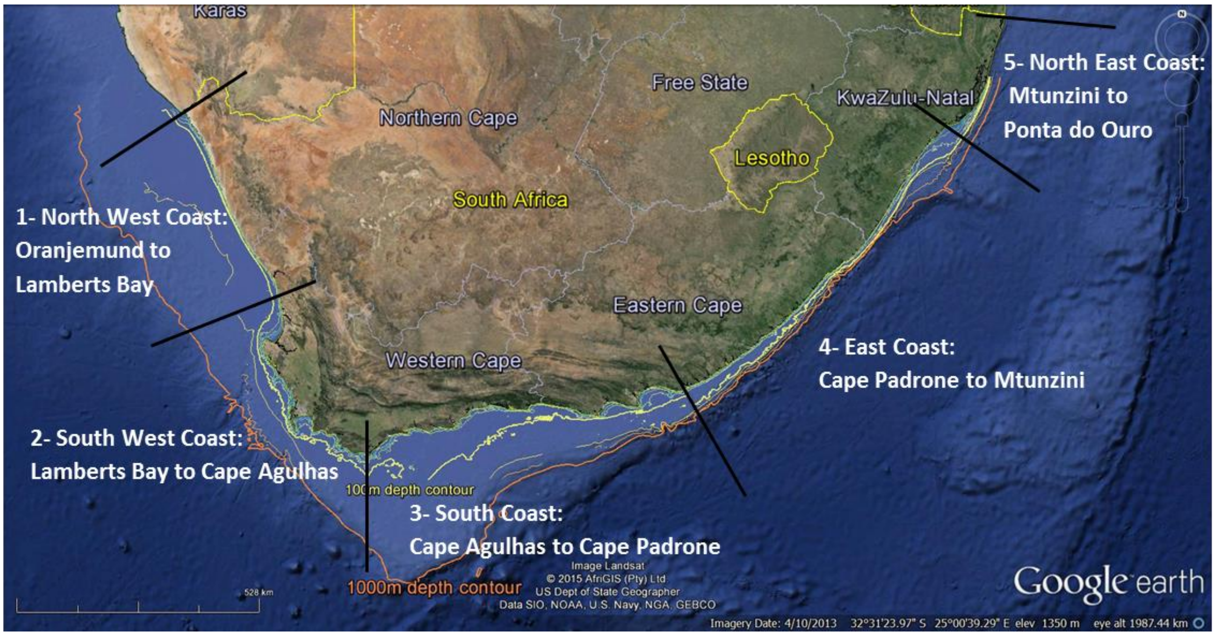


Figure 2.9: South African coastal regions, based on morphological characteristics, general orientation and wave exposure.

North West Coast:

This 420 km of coastline stretches from Oranjemund on the SA/Namibian border to Lamberts Bay located about 200 km north of Cape Town. This is generally a very linear open coastline lacking major capes or headlands, and having only relatively few small bays or coves of very limited indentation. It mainly consists of a mixed sandy and rocky shoreline, as the whole of this coastline is sand starved due to only very few ephemeral rivers feeding into the sea that in addition drain arid inland regions. Due to the predominant south-westerly to southerly offshore wave angles, there is mostly a predominant north-westerly longshore current in the surf zone.

<u>South West Coast:</u>

This 540 km of coastline stretches from Lamberts Bay to Cape Agulhas at the southernmost point of Africa. Within this region the largest bays on the South African coast are found, namely, False Bay, Table Bay, Saldanha Bay, and St Helena Bay, while Walker Bay is less deeply incised. This coastline also faces directly towards the predominant south-westerly offshore swell waves and therefore naturally exhibits large alongshore variations in wave exposure, from the most exposed locations (e.g. at the headlands/capes and open areas) to the most sheltered locations deep inside the bays. Two types of sandy coasts occur most commonly, one being the generally high energy open shorelines often characterised by steeper slopes and more reflectives conditions consisting of medium to coarse sand. The other is characterised by milder slopes and more dissipative conditions often consisting of fine to medium sands, which is typically found in the more sheltered coastal embayments.

<u>South Coast:</u>

This 680 km of coastline stretches from Cape Agulhas to Cape Padrone located about 65 km east of Port Elizabeth. The coastline is characterised by rocky capes interspersed with crenulate-shaped sandy bays (also known as half-heart or log-spiral bays). There are also a few intermittent stretches of rocky and cliffed coastline (e.g. the Tsitsikamma coast). The coastal embayments found in this region e.g. St Sebastian Bay, Still Bay, Vlees Bay, Mossel Bay, Victoria Bay, Plettenburg Bay, St Francis Bay and Jeffreys Bay are less extensive (shallower indentations) than those of the South West coastal region. Only Algoa Bay, which is by far the largest bay along the South Coast, is of the same magnitude as the large South West Coast bays. The South Coast and South West Coast experience the highest incident wave energy along the South African coast. In general the deep-sea wave climate shows a strong predominance of waves (including high storm waves) from the south-westerly quadrant, with a small occurrence of low waves from the easterly sector (Theron and Van Ballegooyen, 2013). Due to the predominant south-westerly offshore swell waves, the bays lie to the east of the capes or headlands and experience predominantly net easterly longshore transport along the more exposed eastern sectors of these bays. Similar to the South West Coast, two types of sandy coasts occur most commonly along the South Coast, one being the generally high-energy open shorelines often characterised by steeper slopes and more reflectives conditions, consisting of medium to coarse sand. The other is characterised by milder slopes and more dissipative conditions, often consisting of fine to medium sands, which in this region is typically found in the more sheltered western sectors of the bays.

East Coast:

This 745 km of coastline stretches from Cape Padrone to Mtunzini located about 35 km SW of Richards Bay. In general, the area is classified as a high-energy environment dominated by south-westerly swells. Although this coast at the large scale, is a generally linear open coastline, the shoreline has an irregular nature with many small headlands and rocky points. Thus it consists of a rocky shoreline interspersed with many coves of small indentation and small pocket beaches mostly located at the river mouths which enter the sea between the headlands or rocky points. There are generally low rates of sediment transport around the headlands and thus also low connectivity between the local coastal sedimentary cells. The northern portion of this region (from about Port Edward northwards) is less irregular and has a mixed sandy/rocky nature underlain by beach rock. According to Palmer *et al* (2011), this portion of the East Coast coast comprises of about 80% sandy beaches, with the rest characterised by intermittent rocky outcrops. The sandiness in this northern portion increases from south to north due to the net north bound littoral drift and the cumulative fluvial sediment contributions. The general shoreline orientation along the central KZN coast is about 30^0 from north (the corresponding shore-normal orientation is 120^0). Theron and Rautenbach (2014) reported that KZN beaches exposed to increased wave action (i.e. located along the open exposed coastline) were found to exhibit larger fluctuations between annual values in beach width (likewise larger vertical differences), compared to those located within more sheltered locations (e.g. within the Durban Bight). Similarly, Cooper (1991b) noted that in analysing shoreline changes along the KZN coast, in most cases the standard deviations are significant, indicating a major influence of episodic events, i.e. the shoreline changes are linked to sea storms.

Typical beach sediments found on the beaches south of Durban have median grain diameters (D50) between about 0.3 mm (medium sand) to 0.9 mm (coarse sand) (Theron and Rautenbach, 2014). An actual net northward longshore sediment transport rate of about 500 000 m^3 per annum (on average) has been estimated along the central KZN coastline by Schoonees (2000), which lies within a most probable range of 450 000 m^3/yr to 550 000 m^3/yr (Theron and Rautenbach, 2014). Based on wave and sediment transport modelling Theron and Rautenbach (2014) found that the annual longshore transport rate towards the south is theoretically about half of the transport rate to the north.

North East Coast:

This 270 km of coastline stretches from Mtunzini to Ponta do Ouro on the South African - Mozambican border. This is also a generally linear open coastline lacking any major capes or headlands. It has a sandy shoreline (Department of Environmental Affairs, 2014), with few small headlands, several coastal lakes, many high dunes, and underlying beach rock which is emergent in a few locations. The incident waves are mostly from a south-south-westerly direction (202.5°), while the second most frequent (17.5%) direction is east (90°). The most frequent extreme waves all approach from the southern to south-south-western quadrant. In general, all the data indicate that peak wave periods of 10 s to 12 s occur most of the time which predominantly represent long-period swell conditions (Theron, 2008). The nearshore zone (inshore of the 50-60 m isobath) does not display any large, current-generated bedforms and in view of the high swell regime is wave-dominated (Cooper, 1991; Diedericks *et al*, 2011).There is generally a net northerly littoral drift due to the predominant south-south-westerly offshore swells. Coastal erosion is episodic and strongly linked to sea storms. Cross-shore or on/offshore sediment transport mainly results from (shorter term) changes in the incident wave conditions (Soltau and Theron, 2006). The type of sandy coast that occurs most commonly along this region is the generally high-energy open shoreline often characterised by steeper slopes and more reflectives conditions, consisting of medium to coarse sand. A few smaller rocky capes/headlands are found here, and due to the predominant offshore swell direction, small bays or shoreline indentations lie mainly to the east of these capes or headlands. These areas mainly represent the few semi-sheltered locations along this region.

2.3. Setback lines – literature review and discussion of methods

2.3.1. Review of international literature on setback lines

There are several international publications that provide information on some aspects of coastal development setback lines, some going as far back as the 1970's (e.g. Collier *et al* 1977, and Purpura and Sensabaugh (1974). The most relevant of the international literature are briefly discussed here.

According to Bruun (1988) beach erosion results from any one or more of the following factors;

- The effects of human impact, such as construction of artificial structures, mining of beach sand, offshore dredging or building of dams on rivers;
- Losses of sediment offshore, onshore, alongshore and by attrition;
- Reduction in sediment supply due to decelerating cliff erosion;
- Reduction in sediment supply from the sea floor;
- Increased storminess in coastal areas or changes in angle of wave approach;
- Increase in beach saturation due to a higher water table or increased precipitation;
- Sea level rise.

Houlahan (1989) and Fenster (2006) provide useful comparisons of constitution setbacks to manage development in coastal hazard areas in 29 States in the USA. Fifteen of these States use fixed setback distances, for example:

Delaware	-	23 m to 30 m landward of the seaward toe of the primary dune;
Florida	-	30 m to >300 m from the reference datum (US NGVD);
Alabama	-	37 m to 137 m landward of mean high water level;
Hawaii	-	12 m landward of the storm runup line or stable vegetation line.

Five States use a "floating" setback distance based on annual long-term average recession rates projected from 30 to 100 years, while four States use a combination of both fixed and floating setbacks. In comparing all of the setbacks, Houlahan found three features to be desirable: (1) designate low and high hazard areas, (2) consider structure size in determining the setback distance, and (3) make the setback approach and implementation understandable to the public.

Beaches and Shores Resource Center (2007), Chiu and Dean (2002) and Komar *et al* (1999), discuss the determination of setback lines based on long-term shoreline erosion trends, short-term shoreline location variations, and coastal flooding levels associated with storm surges and waves. Camfield and Morang, 1996, present a broad discussion of the causes or drivers, processes and time-scales regarding shoreline change, while Feagin *et al* (2005) and Callaghan *et al* (2008) also discuss the causes of coastal erosion.

Cambers (1997) provided guidelines for coastal development setbacks in the Eastern Caribbean Islands, as follows:

1) Cliffs (limestone and volcanic): 15 m landward from the edge of the cliff;
2) Low rocky shores: 30 m landward from the natural coastal vegetation line;
3) Small sandy offshore cays: only temporary wooden structures (/development) to be allowed.

In the case of sandy or gravel ("stone") beaches, the guidelines state that individual setback lines should be determined, which should include consideration of:

i) Historical as well as recent beach changes;
ii) Changes in the position of dunes due to a category 4 hurricane (past and expected);
iii) Coastline changes expected due to sea-level rise (30 years);
iv) Influence of offshore features;
v) Coastal geomorphologic and anthropogenic features or interventions;
vi) Planning considerations.

The setback is determined by adding the provisions for beach changes, dune changes, and SLR induced changes. The final setback is then derived by subjectively also adding provisions for the influences of offshore features, geomorphologic and anthropogenic features, and planning considerations.

Daniel and Abkowitz (2005) expanded Cambers' setback guidelines into a GIS-based system which accounts for: the maximum dune or beach retreat during an extreme storm, a dune stability factor indicative of dune erosion and slumping, existing beach or dune retreat/advance, beach or dune retreat due to SLR, and a safety factor to account for the level of certainty.

Jones and Rogers (2002) describe a statistical method to evaluate erosion risk as well as an approach to balance erosion risk with other hazards commonly considered in U.S. building and flood codes. Following on this, Rogers and Jones (2003) describe other issues that can be used to get the most risk reduction from erosion rate based shoreline setback lines, and state that "shoreline setbacks based on a 70-year erosion rate multiplier are recommended as the best goal to balance the erosion risk with flood, wind and snow hazards to buildings".

Sanò *et al* (2011) discuss the role of coastal setbacks in the context of coastal erosion and Climate Change. They provide a synthesis of the research conducted into coastal setbacks for coastal erosion management and climate change adaptation. This is done by analysing the requirement of the protocol, current processes and management practices in two case study areas (Costa Brava Bays in Spain and Danube Delta, in Romania). Celliers (2010) contains a useful summary of setback regulations in eight European countries, which for example have setbacks ranging from 50 m to as much as 3 km inland, as indicated in Table 2.1.

Table 2.1: Summary of setback regulations in eight European countries (Celliers, 2010)

Country	**Definition**
Denmark	300 m inland beach protection zone 3 km development protection zone inland
England	5 m contour line
Finland	100 m strip along coastline (guide only) which can be increased to 200 m. All development controlled by planning requirements.
Germany	50-200 m inland depending on region. Building prohibited
Norway	100 m inland of shoreline. Building prohibited
Poland	"Technical belt" – depending on shore type, e.g. dune shores up to 200 m inland; cliff shore up to 100 m landward of cliff edge. "Protective belt" – 2 km landward from shoreline to act as buffer for technical belt.
Spain	"Easement of protection" - min 100 m "Easement of passage" - 6 m shoreline passage to be permanently clear for pedestrians "Easement of free public access to sea" – coastal access: 500 m vehicles, 200 m pedestrians
Sweden	100 m inland and offshore. Can be extended to 300 m.

In Western Australia (WAPC, 2003) the total setback distance is determined by adding three components:

1. Provision for normal shoreline variability, in particular erosion during sea storms. This is taken as the modelled recession distance for a 1-in-100 year storm, or if no modelling or survey data is available, then a setback distance of 40 m is specified for this component.
2. A setback distance based on annual long-term (40 year) average recession rates projected to 100 years. In areas where the shoreline is considered to be relatively stable, a minimum setback of 20 m is specified for this component.

3. Setback provision for SLR. This is based on Bruun's rule (see Section 2.6), but with simplified assumptions and a low SLR projection, yielding a distance of 38 m for this component.

If the default minimum values specified above are taken, then the total setback distance comes to 98m. These Western Australian setback distances are all relative to a defined line known as the "horizontal setback datum". The "horizontal setback datum" along sandy shorelines (for example) is "the line indicating the landward limit of annual beach change" (WAPC, 2003).

In South Australia (NCCOE, 2004) the setback is determined by allowing for two components:

1. Provision for shoreline erosion over 100 years together with 0.3 m of SLR.
2. Provision for SLR in terms of coastal flooding levels. This is based on SLR projections of 0.3m by 2050 and 1 m by 2100.

It is interesting to note that New Zealand takes a relatively conservative 1-in-150 year flooding level into account in terms of coastal processes and governance planning, while most other countries apparently deem 1-in-30 to 1-in-100 year as adequate (New Zealand Local Government Guidance Manual, 2014).

Some of the international studies provide useful guidance on specific aspects of setback lines, for example on cliffs, bluffs and rocky shores (e.g. Cambers, 1997), or the Spanish regulations on public access (e.g. Celliers, 2010). These aspects are pertinent to additional components of setback lines in the South African context as well, and are thus discussed in more detail in Chapter 8. However, the international approaches to determining setback lines are generally surprisingly unsophisticated. There is also no apparent preferred approach or consistent methods that are common to the majority of the studies. Therefore a preferred approach for South Africa, built on consistent methods, was developed in this research (Chapter 9). Regarding methods (or models) to determine specific aspects of setback lines, the pertinent literature findings are discussed in detail within the relavant chapters (e.g. wave runup models in Chapter 5).

2.3.2. Review of South African literature and discussion of setback line methodologies

Western Cape setback line methodology (2010).

The Department of Environmental Affairs and Development Planning published a proposed methodology for the determination of coastal setback lines in the Western Cape (DEAD & P, 2010;

Smith 2010; Smith 2011), which procedure was then applied in two example case studies (DEAD & P, 2010: Vol. 2, Appendices D and E). This method relies heavily on the application of several separate numerical models (and on combining their outputs). Hydrodynamic wave modelling, such as SWAN (Booij *et al,* 1999), is for example used to transform offshore wave data to inshore conditions. These inshore wave conditions are used as input to cross-shore morphological modelling (SBEACH; Larson and Kraus, 1989) to simulate storm erosion of the shoreline. Some of the main data requirements therefore include extensive offshore wave data, a time series of measured water levels (including tides and surge), seafloor and inshore bathymetry, and detailed onshore topography.

In general, the method components are conventional and at least adequate to conduct detailed setback studies in small coastal study areas, and could theoretically be applied in this manner in any of the South African coastal regions (based on the characterization of the South African coastal regions in Section 2.2). However, these “detailed” methods rely on comprehensive input data, which is largely not available in South Africa. These “detailed” methods are also time consuming, require data that is expensive to acquire, and cannot practically be “rolled out” to large study areas.

Other aspects which are not entirely satisfactory are as follows:

- It is assumed in this study that if a time period of X years is indicated (e.g. 1-in-50 year setback line) then a 1:X year storm should be considered. However, a 1-in-X year storm does not equate to a 1-in-X year erosion event/distance, e.g. a less intense storm but with longer duration could result in more erosion. Nevertheless, this assumption may be adequate in terms of the purposes of this project.
- It is stated that setback provisions for other issues such as aesthetics, biodiversity and heritage may allow for limited development. However, a convincing argument has not been made for such “concessions” to development to be made. These issues/objectives are not necessarily of lesser importance than physical damage to structures.
- It is stated that model results (from cross-shore morphological models) must be verified against recorded erosion, to allow calibration of model parameters. As a general rule, it is agreed that numerical models should be verified and calibrated. However, this proposed step is problematic, in that recorded erosion data is not available for most South African locations; thus, this step is generally not viable. In fact, the accuracy of the cross-shore modelling is dependent on the accuracy of the wave modelling, which requires detailed inshore bathymetry to enable such accuracy. Such detailed inshore bathymetry data is virtually only available at South African ports and harbours and would be prohibitively expensive to acquire for the larger part of the South African coastline (including most urban and important rural areas).

- No methods are described for determining setback distances from estuary or river mouth channels. A clear method description (which produces quantified outputs) is required.
- It is stated that setback to allow for estuary mouth meander, should be determined from historical aerial photograph assessment. However, no method for calculating this setback distance is provided or described.
- It is stated that calculation (modelling) of wind-blown sand transport potential must be carried out. However, it is considered that quantification of wind-blown sand transports is not essential to determine setback lines. A site assessment, knowledge of local conditions and experience of the practitioner should be sufficient to specify a required vegetation buffer zone width.
- The recommended procedure to assess wave runup does not include the increase in water level due to sea level rise, which is considered to be a deficiency or omission. Furthermore, in applying the procedure in the two example case studies (DEAD & P, 2010: Vol. 2, Appendices D and E), no setback provision for coastal flooding was applied, although the stated procedure makes provision for this.
- The establishment of the setback provision for aesthetic features does not mention who should address this issue or how this setback distance should be quantified, which is considered to be a deficiency or omission.
- According to the report, an attempt was made to simplify the approach and to classify coasts such that standard setback distances could be applied, even as a "rapid assessment" initial setback, but that it was found that no such approach exists. I would contend that there is clearly a very strong need for quicker and less costly methods, and that acceptable alternative measures can indeed be found (as proposed in this thesis).
- The opinion is expressed that the methodology would apply to new developments, while relocation of existing development is deemed unlikely. I would agree that relocation would indeed (perhaps often) involve complex and expensive legal issues. In fact, protective measures (whether natural, "soft" or "hard" structural) are likely to be put in place to protect high value or critical coastal infrastructure or development in situ. However, it seems that in the longer term, relocation will in practise become unavoidable in some instances due to expected climate change effects.

DEADP (2010) includes a useful list of requirements for ideal setback line methodologies:

- "The methodology should be applicable in all 4 coastal provinces. Therefore the methodology should consider conditions prevalent in all provinces;
- The methodology should be generally conservative in considering the accuracy of data, methods and climate change;

- The methodology should not rely on excessively expensive and time-consuming data collection and should minimise costly specialist expertise, over and above the essential coastal processes expertise required;
- The methodology must represent international best practice;
- The methodology must be legally defendable and must withstand legal scrutiny;
- The methodology must ideally be reproducible, i.e. if conducted by another professional a similar result should be obtained."

City of Cape Town's combined risk assessment and setback line method (2012).

The City of Cape Town (CoCT), has determined a management zone, the Coastal Protection Zone (the CPZ, as per the definition from the ICM Act), and a setback line called the Coastal (or Urban) Edge Line (City of Cape Town, 2012a, 2012b; Colenbrander 2010; Colenbrander 2011). "The two coastal zones determined by these lines (i.e. between the HWM and Coastal Edge Line, and between the Coastal Edge Line and the CPZ) are then managed in a manner appropriate to the level of existing or desired development through means of zoning schemes" (adapted from Van Weele *et al* 2014 and City of Cape Town, 2012a, 2012b). Specifically, general zoning schemes are used to determine a base management system of land-use decisions to control various developments/constructions, and then 'overlay zones' are superimposed on the baseline plan to increase or decrease the level of regulation, depending on the assessed coastal biophysical and socio-economic risk. Each coastal management zone is assigned specific regulatory requirements based on land use that include resilient building designs, setbacks that lay out the distance that development must be from shorelines, and natural buffers (adapted from Van Weele *et al* 2014 and City of Cape Town, 2012a, 2012b).

The CoCT approach or "method" is very different for developed and undeveloped coastal areas. In developed coastal areas, the setback line is taken as along the seaward edge of the existing infrastructure, irrespective of whether this development is located within the dynamically active coastal zone (i.e. subject to coastal processes and potential related impacts). An example is given in Figure 2.10, where the CoCT setback line is located along the seaward edge of the existing infrastructure on the northeastern side of the Hout Bay beach, despite it being well known that this development is subject to direct attack from the sea (which occurs during most winter seasons). One reason for this seems to be the desire to avoid or reduce socio-economic "pressures" that are brought to bear and especially legal challenges that typically arise once (proposed) development setback lines are brought to public notice. In CoCT's own words: "whereby the setback, as a means to avoid the legal implications that may arise as a consequence of the position of a setback in relation to private properties with development rights, is defined on the (seaward or) estuary side of the cadastral boundary of those properties. Whilst this approach avoids the legal difficulties, properties located landward of the setback adjacent to (shorelines or) estuaries may still be at risk to …. erosion, storm surge induced flooding etc.". In my opinion this approach is flawed, as the setback line should be based on the actual coastal processes and dynamics, thereby determining which areas are subject to the hazards or where the risks from impacts due to coastal/marine hazards are unacceptably high. The fact that existing development may fall within this area, then actually clearly points out that this is a problem area that needs to be managed. Locating the line seaward of such development can give the false impression that such development is not at risk from the sea or could even be disingenuously used to justify such or similar unwise or misinformed developments. The setback line should thus be correctly based on the actual physical processes, and then the "problem" cases or areas should be dealt with appropriately or managed, which are likely to include socio-economic and legal aspects. This can be illustrated by means of a related example: when say a 100 year flood-line is determined along a river course, this line is not shifted to accommodate a low-lying structure which may be located below this flood-line. (These views are provided although socio-economic and legal aspects are not within the scope of this thesis, as stated.)

Figure 2.10: Position of the draft setback line in Hout Bay as per City of Cape Town *(CoCT, 2012a)*

The CoCT documents also state that due to the technical difficulties of quantifying physical coastal processes, the lines are “fuzzy, and consequently they rather apply zones. Actually this does not seem to be helpful. The mapped zones must have physical edges (lines), which then ultimately lead to the same arguments and problems that supposedly could be avoided by applying zones.

In terms of other technical issues, it seems that coastal flooding hazard has been assessed, but it appears that a setback provision (offset) for erosion (due to sea storms) has not been specifically determined. Some Cape Town beaches experience large fluctuations and if not allowed for, this can have serious consequences. SLR also appears not to be included in the final determination of the Coastal Edge Line and Coastal Protection Zone, which omission is considered to be flawed.

The City of Cape Town’s combined risk assessment and setback line method could potentially be applied to South African cities in any of the coastal regions, but due to the significant problems discussed before, is deemed wholly usuitable for addressing the geophysical coastal-marine processes components of coastal setback lines. The CoCT approach does appear to be more focussed on the social components of setback lines, but this is outside the scope of this thesis.

West Coast and Overberg districts setback line methodology (2012-2013)

Breetzke (2011), Breetzke *et al* (2012), Mather (2011) and Van Weele *et al* (2013) published coastal setback line studies conducted in the West Coast and Overberg districts respectively, both on behalf of the Western Cape Department of Environmental Affairs and Development Planning. Essentially the same methodology was employed in both these studies by mainly the same authors. Therefore, these two references are discussed together.

The authors point out that in terms of the Western Cape Province's initial Coastal Development Setback Lines methodology (DEAD & P, 2010, as discussed in the previous section), two coastal setback lines are envisaged:

- "A physical/coastal process (or hazard) line. This line is proposed to define the limit of the coastal area seaward of which any development is likely to experience unacceptable risk of erosion, flooding by wave action and/or unacceptable maintenance of wind-blown sand accumulations.
- A management (limited/controlled development) 'setback' line. This line is proposed to define areas where some limited and/or controlled development may occur that accommodates requirements of biodiversity, heritage and other aspects not related directly to physical coastal processes. This line is situated on or landward of the coastal processes line."

Breetzke *et al*, 2012 and Van Weele *et al*, 2013, basically use a similar definition of two different types of setback lines. The methods they applied for determining the physical/coastal process line are relatively straightforward and consist of the following steps:

- Determine the 1:10. 1:20, 1:50 and 1:100 year storm off-shore wave height;
- Determine the HWM (and storm runup elevations) based on two wave runup models (one for rocky areas and one for sandy shorelines);
- Determine the short-term storm erosion risk (i.e. short-term cross-shore storm erosion potential) along the coastline;
- Determine the predicted future shoreline regression due to sea level rise;

- Determine long-term beach retreat (i.e. historic trends projected into the future) due to natural sand movement;
- Determination of a final physical processes line (i.e. combination of the above).

During the first of these two studies (covering the Overberg district), overwhelmingly negative public response was generated focused mainly on the impacts of the study on property rights, followed closely by comments regarding stakeholder engagement, amendments to the study and knowledge gaps. A big advantage of the methods applied is that they can be applied relatively easily and quickly, even (arguably) to large (regional) study areas. However, there appear to be certain technical shortcomings (other than mainly perceived socio-economic and legal concerns as mentioned above), which include:

- The model used to calculate wave runup for sandy shorelines, is that developed by Mather (Mather *et al*, 2010, 2011). The Mather *et al* model uses distances offshore x_h to water depth h to estimate a near-shore profile slope as $S = h/x_h$ where the depth of closure is the suggested choice for the water depth h (nominally taken to be about 15 m). Extreme runup R_x is then expressed in terms of S as $R_x /H_0= C.S^{2/3}$. R_x is the runup value, x_{15} is the chart distance from the shoreline to the 15 m isobath, H_0 is deep water significant wave height. In the equation as $R_x /H_0= C.S^{2/3}$, C is a dimensionless coefficient (ranging from 3 to 10) that is used to predict wave runup based on 3 different coastline types (open coast (C=7.5), and large (C=5), or small (C=4) embayment; Mather *et al*, 2011). Although, it appears that this model has produced good results in certain instances (mainly the extreme 2007 storm in vicinity of Durban), it can be said that it's general (South African wide) accuracy (reliability) has not yet been proven (especially where no calibration data exists). In applying the model, the value for coefficient C would be based on relatively subjective selection of either an "open coast" (C=7.5), or "large" (C=5), or "small embayment" (C=4). The total range of coefficient C (from 3 to 10) is wide and implies that results may differ by up to a factor of about 3 if there is uncertainty about the applicability of the coastal type ("open coast", "large -", or "small embayment"). The City of Cape Town (CoCT 2012b, - Annexure D: Addendum to Phase 5: Comparative Study) has for example used Hout Bay and other Cape Town locations as test sites for this model and have pointed out significant variations (up to about 2 m) between the predicted wave runup and actual wave runup measurements in these instances. The runup model does not specifically take account of upper beach slope (or the presence or lack of dunes) which parameter also affects runup elevation according to many authors (e.g. Battjes, 1974, Nielsen & Hanslow, 1991), although a few other authors (e.g. Douglass, 1992) argue that the runup is independent of the beach slope. (The fact that the model does not require the beach slope can be seen as an advantage in that less input data is required making it easier to apply the model to large (regional) study areas.)

- The determination of the short-term cross-shore storm erosion potential along the coastline, is overly simplistic. A fixed 20 m offset (erosion setback) is simply assumed along the entire sandy study area (i.e. excluding rocky shorelines), which does not account for any other alongshore variation in geo-physical characteristics or coastal processes/dynamics (e.g. wave exposure/shelter, presence or lack of dunes, etc.). Such factors can have a significant effect on the magnitude of cross-shore erosion experienced during sea storms (as discussed further in Sections 6.2 and 7.1).
- The determination of the potential long-term beach retreat is based on analyses of historic trends determined from aerial photography, which is acceptable standard practice. However, it seems that as little as four incomplete aerial photography sets were used in the Overberg study, while the photo sets covering the West Coast study area were reportedly of too poor quality to do any shoreline trend analyses. This is considered to be insufficient to derive a robustly defendable trajectory of long-term shoreline change or trends (and unacceptable in case of the West Coast study where this aspect was not accounted for at all). Ideally ten sets of aerial photographs (or more) should be used, which, based on experience, should be achievable in most parts of the South African coast.
- Regarding setback lines around estuaries, the 5m MSL contour is simply designated as the setback line (or the +10 m MSL contour to allow for SLR and a vegetation buffer zone). Whilst this may arguably be a practically acceptable or expedient standard, ideally a more accurate approach that accounts for the different estuarine characteristics and environments should be sought.

The methodologies appied within the West Coast and Overberg districts setback line approach are technically not region specific and should be applicable to any of the South African coastal regions. However, based on the shortcomings detailed before, only some aspects of the approach are deemed suitable for a nation-wide approach and are further assessed (mainly in Chapters 5 and 6). (It should be noted that practically some of the shortcomings may be partially due to the scope of work and available time or funding as specified in the tender documents for these two studies.)

A risk setback line approach for KZN (2013)

Although there are several relevant international publications (as described in Section 2.3.1), only a single peer reviewed journal publication by an South African author about determining or applying setback lines could be found, namely Goble and Mackay, 2013. Goble and Mackay (2013) have presented a method (“process”) to determine a “risk” setback line for KwaZulu-Natal. The authors state that: “the process is simple, cost-effective and considers three key factors: historical shoreline change, sea-level rise and coastal vulnerability”. They also state that: “the methodology is robust and easily repeatable, and that the delineation and enforcement of risk setback lines is a quick solution to address the pressing problems of coastal KZN”. However, scrutiny of this publication leads to the conclusion that it does not present an acceptable SBL methodology, including in the KZN region for which is was developed. Specific shortcomings and uncertainties were identified as follows:

- The method appears to only consider (long-term) shoreline change and "climate variability factors" - there is no consideration of other important geophysical factors, e.g. short-term storm erosion, flooding due to surges or wave runup, slope instability, aeolian transport. Nor of socio- and environmental factors as prescribed by the ICM Act.
- The long-term shoreline change data needs to be described in more detail, e.g. how many data sets are analysed over what time? (It seems that it might be mainly based on Cooper's old shoreline change data (Cooper, 1991a, 1991b, 1994), which would be inadequate).
- The SLR rate taken is historic and does not allow for expected or projected acceleration. It is also not a conservative assumption or "extreme rate" as stated. More acceptable scenarios would be SLR of 0.5 to 2 m by 2100 (Chapter 5).
- The coastal vulnerability scoring method is unclear.
- In the buffer distance formula the coefficient for RB (the buffer distance) is either +1, +0.5 or -1. In the case of accretion, the coefficient is -1. In this case the first and last terms actually cancel out and only the middle SLR term remains. This appears to be illogical in a physical sense and therefore unacceptable.
- In the buffer distance formula the SLR(t) term is actually a vertical sea-level change rate, but seems to be used here directly as a horizontal distance term. According to the given formulation, for e.g. the buffer distance for 100 years = 100 x 3.75mm/yr = 0.375 m, which is meaningless.
- In the coastal vulnerability results discussion it is stated that: "The maximum risk buffer per year is 2 m". It is unclear how this value is derived at and on what logical basis. (It seems that it is possibly based directly on the maximum negative shoreline change rate = -1.97 m/y, which needs to be justified if correct).

It is stated that the risk setback line can be evaluated on the basis of...."the developer's appetite for risk". In my opinion this is unacceptable, as unscrupulous developers could take the maximum risk (i.e. very narrow buffer zone for biggest financial gains) and then transfer this unacceptable risk to unsuspecting parties (e.g. home-owner or local authority).

Traditional South African setback line methodology (pre 2010)

In the foregoing pages setback line methods or approaches applied (or suggested) in South Africa since 2010, have been discussed. A few journal publications of South African origin do mention setback lines "in passing", and whilst of interest in the broader field of coastal zone management (e.g. Glavovic, 2006), they do not contain any specific focus on the matter or any technical discussion thereof. South African publications regarding coastal erosion include Swart (1974), Moller and Swart (1988), Schoonees and Theron (1995), Phelp *et al* (2009), Breetzke *et al* (2008), Smith *et al* (2010), Cooper *et al* (2013), Corbella and Stretch (2012a, 2012b, 2012d), Mather (2012), and Smith *et al* (2013). None of these, however, address setback lines. Regarding setback lines in the pre 2010 period, several South African public domain documents and "grey literature" documents have been published in the form of client or project reports (some of which are not publically available), for example, CSIR (1991), Theron (2003a, 2003b) and Schoonees, *et al* (2005). Based on the pre 2010 South African literature and documents, the setback line methodology mostly applied in South Africa up to 2010, entailed the following steps (in brief):

1. Quantify short-term shoreline variations and calculate the 1:50 year (storm) erosion setback.
2. Identify and quantify possible long-term trends in the location of the coastline. Extrapolate such trends for 50 years and add to the erosion setback.
3. Determine the final coastal development setback by combining the above factors, as well as additional considerations for:
   - a vegetated buffer zone (aeolian sand transport considerations);
   - steep slopes (dunes or bluffs);
   - nature of the shoreline;
   - sea level rise. (Specific methods, sometimes two alternatives, were used to address this, usually by application of Bruun's rule (1983, 1988) to estimate the effect of sea level rise.)

These components are largely generic and have indeed been applied in all of the South African coastal regions. Significant shortcomings of this method include: no consideration of storm surge or wave runup, disregarding the effects of dunes in limiting coastal erosion, and mostly very little or no consideration of ecological or social components of setback lines. Only some of the basic consepts are deemed suitable to apply in a nation-wide approach and are further discussed mainly in Chapters 5, 6 and 8.

Other or supplementary setback line methods occasionally applied in South Africa up to 2010, are as follows:

- Estimation or determination of maximum erosion due to a specific storm. This is usually modelled (mathematically), but may be adjusted according to observed storm erosion. In more comprehensive investigations, a whole range of conditions is simulated or representative storms simulated. Results are then directly analysed statistically or shoreline variation is estimated from the modelling results (e.g. Schoonees, *et al* 2008).
- Comparison of existing shoreline to log-spiral bay “prediction”. This method is only applied in “headland bays”.
- Mathematical modelling of shoreline evolution (1-line) due to effects of structures (e.g. breakwater) or sand pumping (e.g. Soltau and Theron, 2006). In this instance, the shoreline location is directly related to changes in longshore sediment transport.

From the literature and the problems noted in the introduction (Section 1.1) some “top level” requirements of setback lines can be identified:

- Methods for setback lines need to be robust and appropriate;
- Setback line methods must be practical and implementable (on regional or national scale), thus affordable and efficient;
- Methods should be standardized as far as possible;
- A uniform, holistic and integrated approach is required.

2.4. Coastal hazards and risk assessment

2.4.1. Focus of coastal risk assessment versus setback lines

A classical risk assessment framework (following ISO standards) has been adapted to coastal management by Rollason *et al*, 2012 as outlined in Figure 2.11. This framework can be used to indicate the different focus areas of coastal risk assessment versus setback lines, as well as the significant overlap that exists. Thus, the primary focus of risk assessment is on risk identification, analyses and evaluation (as indicated in Figure 2.11). The determination of setback lines also involves risk analyses and evaluation, but the application of setback lines constitutes a risk treatment option (another being, for example, coastal protection structures), and their implementation is an important coastal management strategy, (as indicated by the area within the blue dashed line in Figure 2.11).

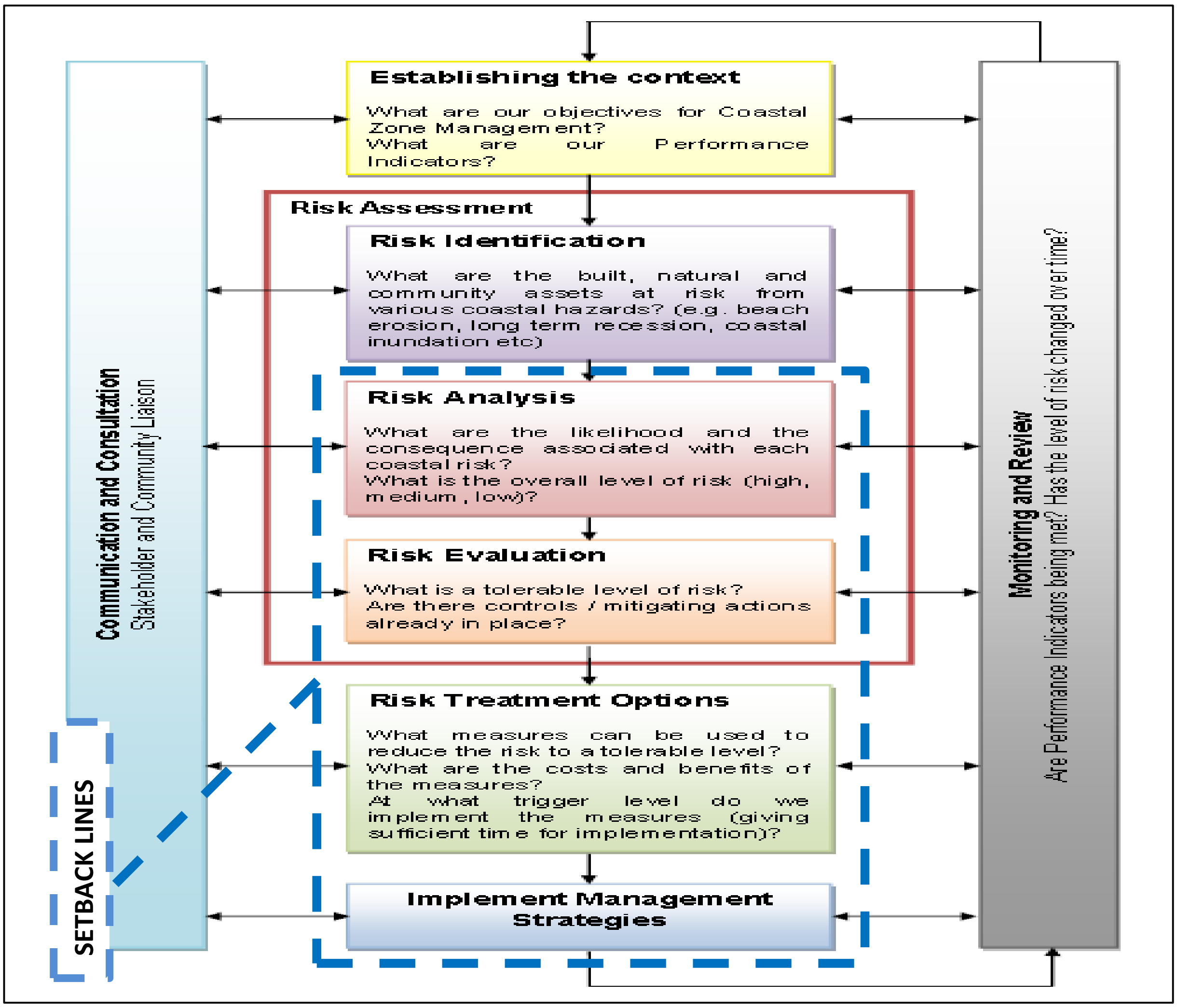


Figure 2.11: ISO 31000:2009 risk assessment framework adapted to coastal management *(from Rollason et al, 2012 with additions by the author)*

Celliers *et al* (2009) have published a useful guide to the ICM Act, in which they state that the purpose of coastal setback lines is to:

- Protect private and public coastal property, including the natural environment;
- Demarcate safe areas, enable definition of areas at risk of being eroded or impacted by coastal processes, and enable the identification of infrastructure vulnerable to the effects of SLR and inundation due to wave runup;
- Achieve conservation and sustainable development; and to
- Achieve other ICM considerations, e.g. bio-diversity, coastal conservation, etc.

The relevance of coastal hazards, spatial vulnerability and risk assessment to setback lines is discussed in more detail in Section 4.1.1.

2.4.2. Relevant abiotic/geophysical coastal hazards and drivers

Van Ballegooyen *et al* (2003) identified all significant marine hazards relevant to parts of the South African coast. A hazard is defined here as an event or process (natural or anthropogenic) that results in a potentially deleterious impact on a desirable status quo. Marine hazards may be due to natural events or anthropogenic activities but are typically a combination of these two causes. Van Ballegooyen *et al* (2003) point out that the full extent of risk (e.g. loss of life and financial loss) is not always fully appreciated, and cite as an example the long-term financial losses due to coastal erosion, which are often poorly understood by both local authorities and land owners. It can be said that all of the items in the hazard inventory of Van Ballegooyen *et al* (2003) result from one or more of the following: erosion, under-scouring of foundations and structures, coastal flooding and inundation, direct wind and wave impacts (occasionally currents), and, broadly speaking, harmful algal blooms and pollution (e.g. oil). (Various threats affecting marine biodiversity in the South African coast and adjacent waters are described and assessed in Taljaard *et al*, 2008.) Leatherman *et al* (2000) states that great sea storms can cause more erosion in a few hours or days than may have occurred in the previous half century (which is usually followed by a subsequent partial or full recovery process lasting up to a decade plus).Titus (1998) points out that a good method to solve beach erosion problems is to put setback lines in place, thus moving development further landward and at a safer distance away from the sea.

Focusing on the abiotic hazards to infrastructure and developments in the coastal zone, one sees that the main metocean drivers are thus waves and seawater levels (and to a lesser extent winds and currents in some instances). This is generally confirmed by literature reviews of coastal vulnerability assessment methods according to which the identified indicators almost all relate to parameters that affect vulnerability or resilience to erosion/under-scouring and flooding or inundation (Theron *et al*, 2010a). The foregoing relates to present hazards, while future hazards related to climate change are discussed elsewhere (Sections 2.5, 2.6 and 8.2).

2.5. Coastal water level extremes

Extreme inshore seawater levels

Significant drivers of high or extreme inshore seawater levels are tides, wind setup, inverse barometric setup (and other oceanographic effects), wave setup and, in future, SLR due to climate change (Theron *et al*, 2010a, 2012). These drivers all affect the "still-water level" of the sea near the shoreline. According to Benavente *et al* (2006), the super-elevation of the coastal water surface consists of three main components, namely barometric setup, wind setup, and wave setup, not taking into account future conditions (i.e. climate change or SLR). Theron (2007) roughly estimated that in the South African setting during extreme events, these components could each contribute additional amounts (heights) of between about 0.35 m to 2 m to the inshore seawater level, as indicated in Table 2.2.

Table 2.2: Parameters and estimated extreme effects on still-water levels for the South African coast *(Theron, 2007)*

Parameters and effects	Elevations (m to mean sea level [MSL]) and setups (+ m)
Mean high-water spring tide	1
Highest astronomical tide (HAT) (~19-yr return period)	1.4
Wind setup	+ 0.5
Maximum setup due to low barometric pressure	+ 0.35
Wave setup (exposed locations)	+ 1
*Hundred-year SLR	+ 0.5 to + 2 (1 m central estimate)

* Note, these estimates for future SLR have been updated from Theron (2007) – see Section 5.2.4.

It should be noted that the above components of extreme inshore "still-water" levels should not be confused with the added effect of wave runup, which can reach even higher elevations. Wave runup is the rush of water up the beach slope beyond the still-water level (i.e. the swash zone). According to surveyed elevations (Smith *et al*, 2010), maximum runup *elevations* on the open KwaZulu-Natal (KZN) coast near Durban during the March 2007 storm (which coincided with highest astronomical tide) reached up to about 10.5 m above MSL. Note that wave setup and runup are both accounted for in these levels. The maximum wave runup height *alone* during the 2007 KZN storm is estimated to have been up to about 7 m (vertical), resulting from significant near-shore wave heights of about 8.5 m (Phelp *et al* 2009). (The horizontal distance that the coastline retreated due to coastal erosion caused by this storm ranged from in the order of 0 m to 100 m, resulting from local circumstances.) In

South Africa, wave runup is thus an important factor (and often dominant), which may be considerably exacerbated by tides and future SLR (Theron *et al*, 2010a). This is in line with findings from the US, where it has been shown that on the US East and Gulf Coasts, storm surge (related to barometric pressure and/or wind effects) is often dominant, while in Southern California (as in South Africa), where the continental shelf is narrow and hurricanes (cyclones) rare, storm surge is often negligible compared with "wave effects" (e.g. Stockdon *et al*, 2006). The 2007 KZN storm (in the order of a 1-in-10-year to 1-in-35-year event; Phelp *et al*, 2009) should at least serve as a timely warning of the potential impacts (Figure 2.12) that could be incurred much more frequently in future or exceeded in any year by more extreme events.

Figure 2.12: Example of the impact of the March 2007 KwaZulu-Natal sea storm *(Photo: D Phelp)*

All of the above mentioned components of extreme inshore seawater levels as determined for the South African coast are discussed in detail in Chapter 5. More accurate region- and scenario-specific quantifications of each of these components are also provided in Chapter 5.

Wave runup

An important step in quantifying coastal flooding levels and in calculating setback lines (i.e. adequate development setback distances), is the determination of wave runup, in other words the maximum point that storm waves can reach. The wave runup is mainly a function of parameters such as wave height, direction and period, the surf zone width, the type of wave breaking, the roughness, slope and permeability of the near- and inshore profile (e.g. rocks or sand), the shape of the beach profile and the wave height distribution (Battjes, 1974). A steeper inshore and beach slope, for example, can lead to more severe wave runup. In a preliminary literature review of wave runup prediction methods, several methods were considered by the author. These are the model of Battjes (1971); that of Nielsen and Hanslow (1991); three formulations by Ahrens and Seelig (1996)); two formulations by Ruggiero *et al* (2001); the model of Guza and Thornton (1982); and that of Stockdon *et al* (2006). More recently a promising formulation for South Africa has been proposed by Mather *et al* (2011), but the general validity and applicability for South African conditions need to be investigated further. Of the more empirical formulations, those of Nielsen and Hanslow (1991) and Ruggiero *et al* (2001) appear to be most suitable, with the former being easier to apply. However, to date the application of current wave runup models are inadequate in that large variations (up to about 2 m) between the predicted wave runup and actual wave runup measurements are often found. Some of the reasons are related to most of the models being semi-empirical (especially the older models, e.g. Battjes (1971), Nielsen and Hanslow (1991), Ahrens and Seelig (1996), Ruggiero *et al* (2001) and Guza and Thornton (1982)), that they have limited applicability and that they typically do not account for some processes which are thought to be of importance. The Nielsen and Hanslow model (1991), for example, is deemed to be inaccurate for 'flat' (low-gradient) beach slopes. Even the promising newer models (e.g. Stockdon *et al* (2006), (Mather *et al* (2011), etc.) have significant shortcomings, for example, the Mather *et al* (2011) model does not specifically take account of upper beach slope (or the presence /lack of dunes) which parameters are known to also affect runup elevation (e.g. Battjes 1971). Even "full process based" models (e.g. Wei *et al* 1995) do not necessarily seem to yield robust results and are impractical to apply in many instances (e.g. Eurotop approach [Pullen *et al*, 2008]). Thus, our current understanding of the relevant physical processes still appears to be lacking, and further research is needed directed towards generating improved quantification of wave runup phenomena to fill this critical gap in determining setback lines.

Coastal flooding levels and probabilities

In the foregoing paragraphs it has been shown that high or extreme inshore sea water levels are due to combinations of some or all of the following drivers: high tides, wind setup, hydrostatic setup, wave setup and runup, and in future, sea-level rise. The joint probability of spring high tides (occurring for approximately say 18 hours in total over 14 days) with a 1-in-100 year sea storm or cyclone (with possible extreme local effect of say 3 days) and a long-term 1 m SLR scenario by 2100, could be more severe and less frequent than a true 1:100 year extreme coastal flooding event. Relatively long-term water level recordings, which include sufficient sea storm or cyclone events and resulting setups, are required to calculate statistically accurate extreme events and occurrences. Unfortunately, such data for South Africa is insufficient; therefore, following the precautionary approach, plausible scenario combinations are currently applied, which is considered a first level approximation. There is presently no validated method to assess the joint probability of tides, surge, runup etc. along shorelines (e.g. Alcock 1993, Lynett *et al* 2009). Typically, the different components of high inshore sea water levels due to tides, wind setup, hydrostatic setup, wave setup and runup, and sea-level rise (SLR) are computed and combined in a rudimentary way or by considering various limited scenarios. However, it is known that some of these phenomena are indeed often inter-related (not independent), and also that no accurate recurrence levels can be attributed to such "rudimentary" combinations of events (e.g. Alcock 1993). Research relating to inshore seawater levels is required to better understand the relationships between metocean events and physical coastal processes, and how joint occurrences affect extreme coastal flooding levels.

2.6. Climate change

An additional geophysical coastal threat (besides those described in Section 2.4), which is at present finally being generally recognised at many levels of society (public and private), is that posed by global warming (Stern, 2006). (The awarding of 2 Oscars to the "Global Warming documentary" film made by Al Gore, is another case in point.) Since the 1970s the "greenhouse effect" and sea-level rise have continued to generate interest and concern. Coupled with this have often been dramatic predictions of massive coastal impacts (e.g. Hughes and Brundrit, 1990). Global average eustatic or absolute SLR is mainly due to a combination of an increase in ocean volume due to lower seawater density, arising from a warmer ocean temperature and lower salinity, and an increase in ocean mass due to a redistribution of fresh water from land-based storage (e.g. glaciers, ice sheets, dams, lakes, rivers and groundwater) to the oceans (Ministry for the Environment, New Zealand, 2008). Thus, the sea level rises when meltwater from land-based masses of ice, such as glaciers, flows into the ocean, but the sea level also increases when heat from the atmosphere is mixed into the upper layers of the ocean, causing that water to expand. In recent decades, this thermal expansion has caused, on average, only about one quarter of the SLR seen each year, but its contribution is increasing (Gillett *et al,* 2011). Researchers are now pointing towards an even bigger threat from warm ocean waters: the floating ice shelves that ring Antarctica could melt, and so could the seaward end of land-based ice streams, which would lead to a long-term, catastrophic rise in sea level (Gillett *et al*, 2011). In combination with other factors, such as subsidence and glacial isostatic adjustment, SLR relative to the land will be highly localised (PIANC, 2008). At mid latitudes the mean SLR will be generally higher than in the equatorial area (IPCC, 2007; Church *et al*, 2004) due to changes in ocean density distribution (steric SLR).

The National Committee on Coastal and Ocean Engineering of Australia (NCCOE 2004) identified a number of potential major impacts for the coastal zone resulting from climate change, such as:

- inundation and displacement of wetlands and lowlands;
- eroded shorelines;
- increased coastal flooding by storms;
- salinity intrusion of estuaries and aquifers;
- altered tidal ranges, prisms and circulation in estuarine systems;
- changed sedimentation patterns;
- decreased light penetration;
- changed storm patterns, windiness, wave energy or direction impacting coastal stability and alignments.

Climate change is also expected to have a number of other consequences which will detrimentally affect coastal resources. These are (amongst others): higher sea levels; higher sea temperatures; changes in precipitation patterns and sediment fluxes from rivers; changed oceanic conditions; as well as changes in storm tracks, frequencies and intensities. The apparent increase in storm activity and severity will be the most visible impact and the first to be noticed, since higher sea levels will require smaller storm events to overtop existing storm protection measures (Theron, 2007).

The potential specific impacts of sea-level rise in terms of shoreline recession, have also been considered (e.g. Bruun, 1983, 1988). In the UK, for example, CC scenarios looked 30-80 years ahead; even in that timescale, damage due to coastal erosion is set to increase by 3 to 9 times (Allsop, 2005). Climate change (CC) and sea level rise potentially also have far-reaching consequences for South Africa's coastal provinces where the great majority of the population live and work in, or near, the coastal zone (Midgley *et al*, 2005). Due to such impacts and the uncertainty of sea-level rise predictions, more comprehensive studies into the potential effects and impacts are required (IPCC, 2001, 2007).

The problem with SLR is not just the vertical rise but also its interaction with changing storm intensities and wind fields to produce sea conditions that will progressively overwhelm existing infrastructure (e.g. Battjes, 2003; Houghton, 2005). These interactions pose a particularly important risk in the case of the highly exposed South African coastline, and is a subject that up to now has been little explored, even internationally. Although we are not at this time able to reliably estimate changes in storm patterns, windiness, wave energy or direction, the increase in storm activity and severity will probably be the most visible impact and the first to be noticed (Theron, 2007). Thus, locally applicable methods have to be developed urgently to account for the impacts along the South African coast. The knowledge gained from this research should be incorporated within specific coastal management strategies, as emphasised in the new South African environmental and coastal legislation (Theron, 2011). To mitigate the detrimental impacts of climate change, we have to understand the adaptation options available to South African society (Glavovic, 2000), which is considerably different from 1st world approaches, and still largely unexplored (Theron, 2007).

In South Africa, some good research has been done on sea level rise (for example, Brundrit 1984, 1995; Hughes *et al* (1991), Mather, 2008, Mather *et al*, 2009, Mather and Stretch, 2012). Coastal climate change effects and related potential issues in South Africa are discussed in Brundrit, 2008; Cartwright, 2008; Cooper (1991a, 1991b, 1994, 1995a, 1995b), Fairhurst, 2008; Mather, 2012, Midgley *et al* (2005), Theron (1994, 2007, 2011). Internationally, there is a huge amount of literature available broadly related to Climate Change, but regarding specifically the interaction of sea level rise with changing storm intensities, there are very few publications (especially South African publications), one local exception being Theron *et al* (2008). A good general knowledge basis has been laid regarding CC issues in South Africa, but there is thus still a dire need for improved understanding of, and especially predictive capabilities regarding the interaction of sea level rise and increased storminess on coastal erosion.

Future scenarios for projected climate changes and coastal drivers relevant to setback lines (for example, sea-level rise) are discussed in detail in Sections 5.2.4 and 9.1.

2.7. Long-term trends in South African wave climate

Preliminary findings indicate that there may be long-term trends in regional marine weather (metocean) climates, while SLR alone will greatly increase the risks and impacts associated with extreme sea storm events (Theron, 2007). The regional variation in the global wave climate was demonstrated by Mori *et al* (2010), who predicted that the mean wave height might generally increase in the regions of the mid-latitudes (both hemispheres) and the Antarctic Ocean while decreasing at the equator. Their study was based on simulating future trends. Further evidence of a general wave height increase in the northern Atlantic along the North American east coast was provided by Wang *et al*

(2004). Komar and Allan (2008) also found an increase in the wave height generated by hurricanes along the east coast of the United States of America using wave data from the National Data Buoy Center (NDBC wave buoy data). Investigations by Ruggiero *et al* (2010) on buoy data also indicate increasing storm intensities along both the west and east coasts of North America. Such changes in the regional metocean climates are expected to have significant impacts on local coastal areas. It is therefore important to also investigate possible future climatic changes off the South African coastline as well as the expected associated impacts.

As can be anticipated, a more severe wave climate (or indirectly a more severe oceanic wind climate) will have a greater impact on wave runup, coastal flooding levels and erosion, thus necessitating the prediction of future trends in the wave climate. Although the available South African wave record is shorter than ideally required to determine long-term trends, a preliminary analysis was conducted (Rossouw and Theron, 2012). It was found that the annual mean significant wave height (Hm_0) and corresponding standard deviation for the wave data set collected off Richards Bay and the annual mean wave height (Hm_0) for the long-term data set collected offshore of Cape Town indicated no real progressive increase. This may appear to contradict the findings of the Intergovernmental Panel on Climate Change (IPCC) as presented in PIANC (2008). However, the South African results may reflect a regional aspect of the impact of climate change.

Although the averages of the South African data appear to remain constant, the individual storm data shows some change. For example, considering the peaks of individual storms during the more extreme South African winter period (June to August), an increase of about 0.5 m over 14 years can seemingly be observed (Figure 2.13, from Rossouw and Theron, 2012). The trend could potentially be indicative of a significant increase in ’storminess’ over the next few decades, but such a large trend is considered unlikely at this stage. It is also worth noting that the opposite occurs during summer; there seems to be a general decreasing trend over the last 14 years with regard to individual storms.

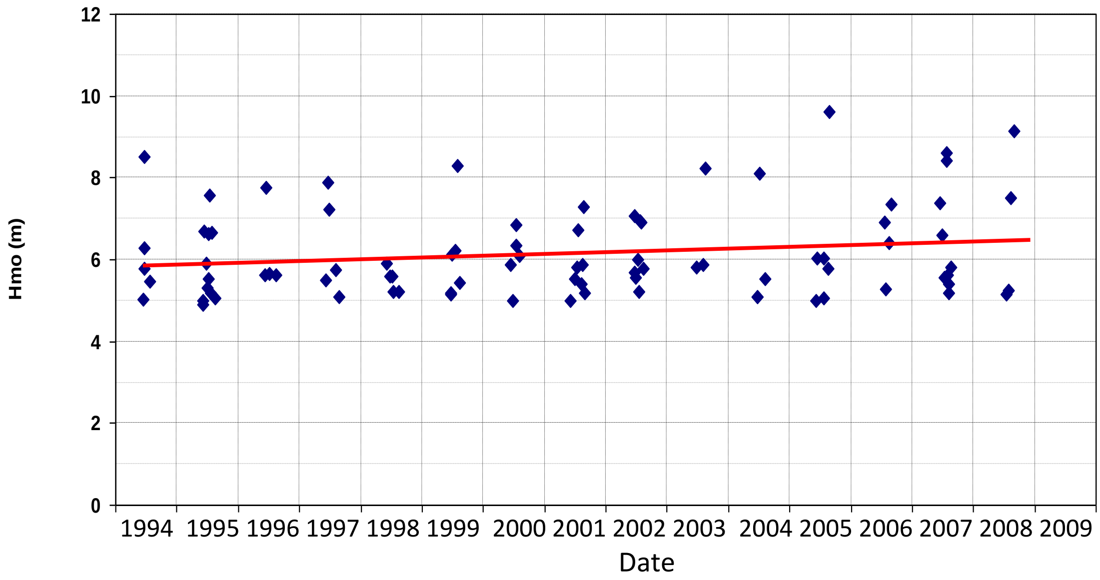


***Figure 2.13: Peaks of individual storms over a 14-year period – offshore Cape Town** (Based on recordings by the CSIR; Rossouw and Theron, 2012).*

If the recorded increase is indeed indicative of a trend, storminess (in terms of intensity) may be on the increase. An extrapolation into the future of the previous 0.5-m wave height increase over 14 years is, however, considered to be unrealistically high. To some extent it could be said that an increasing trend (as possibly indicated by the South African wave data) is supported by the model predictions of Mori *et al* (2010), which appear to show an increase for the southern Indian Ocean of roughly 6% (at exceedance probability < 10^{-5}) (Figure 2.14). This is also supported by the most recent IPCC report (IPCC AR5 SPM, 2013), which states that the mean significant wave height is likely (i.e. 67-100% certain) to increase in the Southern Ocean.

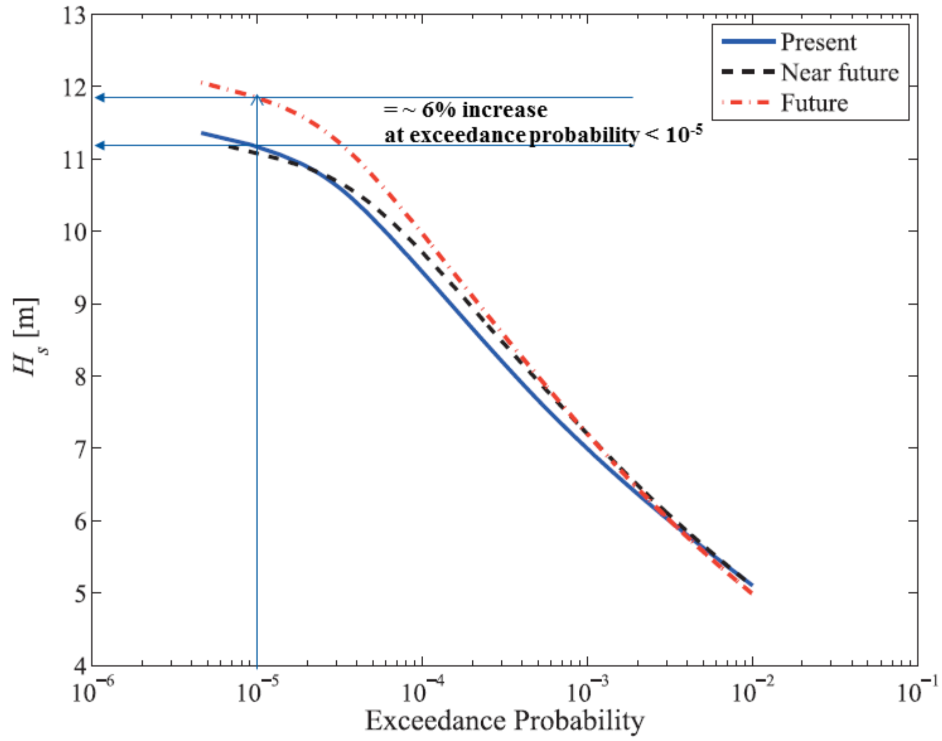


Figure 2.14: Future wave climate changes from model predictions *(Mori et al, 2010)*

Based on 18 years of wave recordings off Durban, Corbella and Stretch (2012c) concluded that maximum significant wave heights (0.03 m/year), average wave direction, peak period and storm event frequencies all show weak increasing trends, but that only the increases in peak period and wave direction are statistically significant. (It seems that the wave data used in this analysis is probably collated from recordings made at significantly different water depths. This significantly increases the uncertainty of longer term trend analyses.) Both the Cape Town (Rossouw and Theron, 2012) and Durban wave data (Corbella and Stretch, 2012c), currently (2014) have insufficient record lengths to identify long-term trends with acceptable certainly.

In lieu of a sufficiently long record of wave data and consequently wave climate trends, the main driver of the waves, namely the ocean winds, can be examined to derive possible trends. Wave climate and conditions are determined by ocean winds (through parameters such as, e.g., velocity, duration, fetch, occurrence, decay and depth). PIANC (2008) suggested that there is no evidence to support designing for any specific change in wind conditions. Based on analysed results for future wind regimes from 10 Global Climate Models (GCMs), Luger (2012) estimated offshore wave heights off Table Bay. His wave change scenarios for 2060 off Table Bay were: best estimate +1%, upper estimate (90th percentile) +10%, but significantly larger increases were predicted for False Bay due to projected increases in southerly and easterly wind components. A 5% increase in storm surge by 2060 was applied in this study. The fact that Luger's study was focused narrowly on specific sites within the Cape Town metropolis, and that the 10 Global Climate Models yielded highly inconsistent results, means that this study is not appropriate or suitable to provide quantitative direction regarding the future regional wave regime off South Africa.

Despite such limited studies and the possibility of stronger oceanic winds off Southern Africa being mentioned in the literature (e.g. IPCC, 2007; Joubert & Hewitson, 1997), predicted values for potential changes in wind regimes off the southern African coastal region are currently still largely lacking. Jury (2013), for example, also mentions both a warming and accelerating trend of the Agulhas current and associated shift in the zonal wind belts, but specific scenarios of changes in oceanic wind velocities are not provided. Rouault *et al*, (2010) show that both South Atlantic and South Indian Ocean trade winds have increased in the order of 0 to 0.4% per decade over the period 1979-2001, but do not provide future projections (during which the changes are expected to accelerate). In view of the dearth of quantitative metocean projections suitable for South African coastal engineering applications and to enable an assessment of the potential impacts of stronger winds, a relatively modest increase of 10% could be assumed. This is also in line with assumptions made for the UK (DEFRA, 2006), the German coast (Brinkmann, 2010) and the Mozambican coast (Theron *et al*, 2012).

The possible implications of a 10% increase in wind velocity is illustrated as follows: Wave height (in the fully developed state) is proportional to the square of the wind stress factor (UA). UA can be related to the wind speed (U) according to the following expression (United States, Army Corps of Engineers, 1984):

$$UA = 0.71\ U^{1.23}$$

Thus, a modest 10% increase in wind speed means a 12% increase in wind stress and a 26% increase in wave height (Theron, 2007).

Regarding cyclone generated waves that occasionally occur along the South African north-east coast, some global climate models seem to predict an increase in frequency and intensification of cyclones (e.g. Carter *et al*, 1994), but there does not seem to be general scientific consensus on such future cyclone changes or trends. Knutson *et al* (2010) predicted a small reduction in the number of cyclones in the Mozambique channel, with approximately unchanged wind speeds, but a 10% increase in pressures. Jury (2013) reported a poleward migration of the sub-tropical anticyclones in southern Africa. While about two to three cyclones per year currently enter the Mozambique Channel, a possible southward shift of the cyclone belt due to climate change would mean an increase in the occurrence of cyclones impacting southern Mozambique's coastal regions (Theron *et al*, 2012). However, although this is a projected future outcome of climate change effects, the confidence placed in this projection is low at this stage. This potential effect of climate change is also not expected to occur within the next few decades but is possible in the long term, perhaps only beyond 2100 (Rossouw and Theron, 2012). On the other hand, Malherbe *et al* (2013) reported that their projections (under the A2 emission scenario) for the latter part of the 21st century indicate "a decrease in the occurrence of tropical cyclones over the Southwest Indian Ocean adjacent to southern Africa, as well as a northward shift in the preferred landfall position of these systems over the southern African subcontinent". It seems that the available literature to date provide insufficient basis on which to amend the wave climate off the east coast of South Africa due to potential future changes in the cyclone regime in the Indian Ocean.

2.8. Conclusions

The South African coastline is rugged and exposed, with few natural bays, and consists of long stretches of sandy beaches interspersed by rocky sectors. For the present study the coastline has been sub-divided into five regions (Figure 2.9), on the basis of their morphological characteristics, general orientation and exposure to waves. The South African coast contains no muddy shorelines (other than inside some estuaries), nor barrier island coasts or delta coasts. Two types of sandy coasts occur most commonly, one being the generally high energy open shorelines often characterised by steeper slopes and more reflectives conditions consisting of medium to coarse sand. The other is characterised by milder slopes and more dissipative conditions often consisting of fine to medium sands, which is typically found in the more sheltered coastal embayments.

Coastal Development Setback Lines are a legal requirement under the South African ICM Act (2008) and a critical component of integrated coastal management strategy. They have a multiple purpose in terms of coastal protection, conservation and demarcation (as detailed in Section 2.3.1).

Some "top level" requirements of setback line methods have been identified (e.g. robust, appropriate, practical, implementable (on regional /national scale), affordable and efficient, standardized as far as possible, and holistic and integrated). This clearly points out the need for guidelines regarding methods, norms and standards for setback lines. Some of these are similar to the requirements listed by DEAD&P (2010), for "ideal" setback line methodologies.

"Erosion and coastal processes" setback lines (mainly for safety and to protect property from abiotic physical coastal/marine processes or "impacts") should include the following basic components:

1. Setback provision for flooding, inundation, direct wave impacts: extreme water levels and wave runup.
2. Setback provision for long-term coastline changes (shoreline location trends).
3. Setback provision for short-term shoreline variation, e.g. erosion due to storm waves.
4. Setback provisions for additional aspects:
   i. potential climate change effects, primarily sea level rise and possibly wave height increase;
   ii. wind-blown sand;
   iii. bluff, dune or cliff instability;
   iv. estuary or river mouth dynamics.

The foregoing aspects together with other ICM considerations and requirements such as public access, biodiversity or environmental conservation, heritage, etc., feed into the determination of coastal development setback lines.

Although some of the international studies provide useful guidance on specific aspects of setback lines, the approaches are generally surprisingly unsophisticated. The specific aspects that are pertinent to the additional (i.e. mainly besides coastal flooding and erosion) components of setback lines in the South African context, are discussed in more detail in Chapter 8. There is no apparent preferred approach or consistent methods that are common to the majority of the studies.

Despite the availability of the literature cited and discussed in Sections 2.1 to 2.7, to date there has been a paucity of studies on shoreline dynamics, erosion and long-term changes on the South African coast, and very few that included integration of these aspects and application in terms of setback lines. This demonstrates that there is a lack of knowledge in the literature in the field of coastal engineering regarding applicable theoretical methods, appropriate guidelines and practical application thereof in terms of setback lines for South Africa.

Detailed and comprehensive investigations have occasionally been conducted to determine setbacks required for local shoreline variations, but then only for small study areas and where extensive input data is available. Two-dimensional profile modelling (time step) can potentially yield good results, but requires lots of input data, verification and calibration; thus, it is only suitable for small study areas, and is time consuming (thus costly) to apply. On large scale studies, it is not practical or affordable to conduct many such detailed local setback investigations.

On the other hand, in some South African studies, a very simplistic approach has been applied. To allow for normal shoreline variability (e.g. erosion during storms and accretion recovery thereafter) a setback distance of 20 m or 40 m is specified (and in some instances added to other setback distances to account for additional factors) to derive an acceptable total setback distance. The 40 m is the distance specified in some Australian (WAPC, 2003) and US states (Houlahan, 1989 and Fenster, 2006). Such fixed offsets do not account for any alongshore changes in shoreline characteristics or in the coastal/marine processes and metocean drivers.

Different approaches are currently being employed within the South African coastal provinces. In the five post 2010 South African studies reviewed, four totally different approaches have been used. The "robustness" of the results differs for the different methods (also for the pre 2010 South African studies), depending also on aspects such as the quality of input data, calibration/verification data, accuracy of the method, etc. Although the methodologies appied since 2010, have each only been applied in specific regions, none of them are actually region specific and could potentially be applied in any of the South African coastal regions. The two approaches that have seen the widest application, are: the West Coast and Overberg districts setback line methodology (2012-2013), and components of the traditional South African setback line methodologies (pre 2010). Based on the characterization of the South African coastal regions (Section 2.2), these two methods are indeed applicable to all of the South African coastal regions, and (following from the specific critique of each method as discussed in Section 2.3) are also considered to be the most suitable contenders for a nation-wide approach. However, while some aspects of these two approaches are deemed suitable and are further assessed (mainly in Chapters 5 and 6), there are also significant shortcomings (as detailed in Section 2.3) rendering these approaches unsatisfactory for general nation-wide application. The mostly applied traditional South African setback line methodologies (pre 2010) also have specific shortcomings, in terms of application of an appropriate "dune methodology", and analyses of historical shoreline variations and trends. Both the literature review and recent setback line workshops held in South Africa have highlighted the lack of consistent methods to determine setback lines as well as the major confusion around how to proceed.

Long-term coastal zone planning, including setback lines, should consider the potential climate change effects of a possible wave height increase. Based on the information and discussion in Section 2.6, it is concluded that the main scenario for future wave climate off the South African coast should be a 6% to 10% increase in wave height by 2100, with the best estimate a 6% increase, as derived from Mori *et al* (2010).

Regarding the potential climate change effects of sea level rise, future scenarios to consider are discussed in detail in Section 5.2.4.

Chapter 3: Approach and research method

3.1. Approach and research methodology

The approach to the research was strongly driven by the problem statement, i.e. there is a need to investigate, develop and define appropriate coastal setback line methods for "data poor" environments, that can be efficiently applied in large South African study areas, but that are still sufficiently robust and defendable. A need was also identified to find ways of addressing specific technical shortcomings in methods that have to date been applied to determine setback lines in South Africa (for example, accounting for dunes). Informed by thorough literature reviews of these topics, and having identified which specific aspects of setback lines therefore needed to be addressed, these were studied in detail.

The general methodology was essentially a comprehensive and intensive desktop research study on all aspects of coastal setback lines, based mainly on analyses and synthesis of available South African data and information regarding geophysical coastal processes and related metocean drivers, but further informed by available literature, site visits, an in depth understanding of local and regional abiotic coastal processes and shoreline dynamics, as well as development of new numerical methods, supplemented by use of existing numerical models. In the literature review it was found that the primary coastal processes components of setback lines concern coastal flooding levels and coastal erosion. Although adequate methods do exist to quatify these components in fine scale, detailed studies in small coastal study areas, there are gaps in terms of methods that can be applied at regional or larger scale and that are realistic despite a paucity of input data. One of the aims of this research is therefore to find or develop methods that address these science gaps, which are still sufficiently robust and applicable to the various South African coastal environments.

Regarding coastal flooding levels, the approach was to firstly determine extreme values for realistic combinations of all the inshore seawater level components, based on analyses of comprehensive South African data. This enabled determination of the regional storm surge levels around the South African coast for the main offshore wave conditions. Secondly, regarding the wave runup component of coastal flooding levels, the approach was to collate all available South African data and to test a wide selection of numerical models from the literature against this data. The best performing models were then tested further and recalibrated where possible or their application method was adapted to yield the best possible results for all of the various South African coastal environments.

Regarding coastal erosion, essentially a down-scaling approach was followed, utilizing semi-empirical relationships. This is a suitable approach because of the typically large spatial scales (from tens of kilometres to a few hundered kilometres) and long temporal scales (from a few decades to more than 100 years) on which the methods need to be applied to. The methods also needed to be suitable for a wave-dominated coast (as South Africa is) considering processes relevant to coastal erosion (waves being the dominant factor in coastline response in this case), with the purpose of predicting the large-scale behavior of the shoreline (in a sense similar to other behavior models such as Unibest-CL, Lipack, Astima and Estmorph (Bosboom and Stive, 2014)). The data poor South African situation necessitates input reduction, which therefore requires process reduction in applicable methods (i.e. model complexity generally has to be reduced for lack of comprehensive input data required to simulate all detail processes). Finally, in view of promulgated time-frames for setting and affordability of determining setback lines in South Africa, applied methods need to be efficient (i.e. large scale applications with modest computational and manpower requirements). Thus, the approach followed was to develop new methods in accordance with the foregoing, and two alternatives are proposed: a statistical and a parametric approach. Although a prerequisite of the statistical model is both process knowledge and data knowledge, this model relies on understanding / predicting the shoreline behavior based on measured data. The basis of the parametric approach is that it should be able to describe the gross cross-shore processes and behaviour of the shoreline based on simplified parameterised functional relationships that reflect the morphologic phenomena on a larger scale (Van Rijn, 1998). (According to Stive and Walstra, 1998, parametric models can also be considered as reduced process-based models, where the dominant processes are modelled by means of parameterization.) The approach followed here is generally not suitable or intended for detailed designs.

Where alternative or improved methods were identified or developed to quantify aspects of setback lines (as discussed above), appropriate and extensive South African geophysical coastal data was collated and analyzed to test these methods. In this manner the suitability and applicability of these methods to South African conditions were verified as far as possible. To demonstrate the use of the methods, and test their veracity, they were also applied in case study areas representative of various/different South African coastal areas. By comparing the new or adapted methods to other more detailed methods and models and especially to field measurements, their robustness and applicability for a variety of environments that are representative of the South African coast are assessed. Based on the foregoing, the required components of coastal development setback lines are given, as well as how these should be determined. Recommendations on both the methods for and the actual determination of setback lines in South Africa, as well as guidance to assist in the application of such methods are made.

A flowchart indicating the path followed in conducting this research and the related thesis content is depicted in the diagram in Figure 3.1. Note that the numbers indicated within the flowchart correspond to the thesis chapter numbers. The flowchart indicates that the background investigation, literature review and study of coastal hazards together informed what the requirements for setback lines are, which components must be included, what the current shortcomings are, and therefore which aspects of setback lines needed to be investigated. These initial studies also all informed the final formulation of the research rationale and setting of objectives. The flowchart further indicates how the subsequent studies of the main components and additional aspects enabled the final compilation of all the steps required to determine setback lines together with the recommended procedures and methods.

The delineations and limitations of this thesis are discussed in Section 1.3, but it is perhaps appropriate to reiterate here that the focus of this thesis is strongly on the abiotic (geophysical) components of setback lines, while environmental and social aspects are also briefly discussed. Although the application of setback lines has a strong legal connotation (according to the South African ICM Act of 2008), the focus of this study was similarly not on addressing the legal aspects (and related problems).

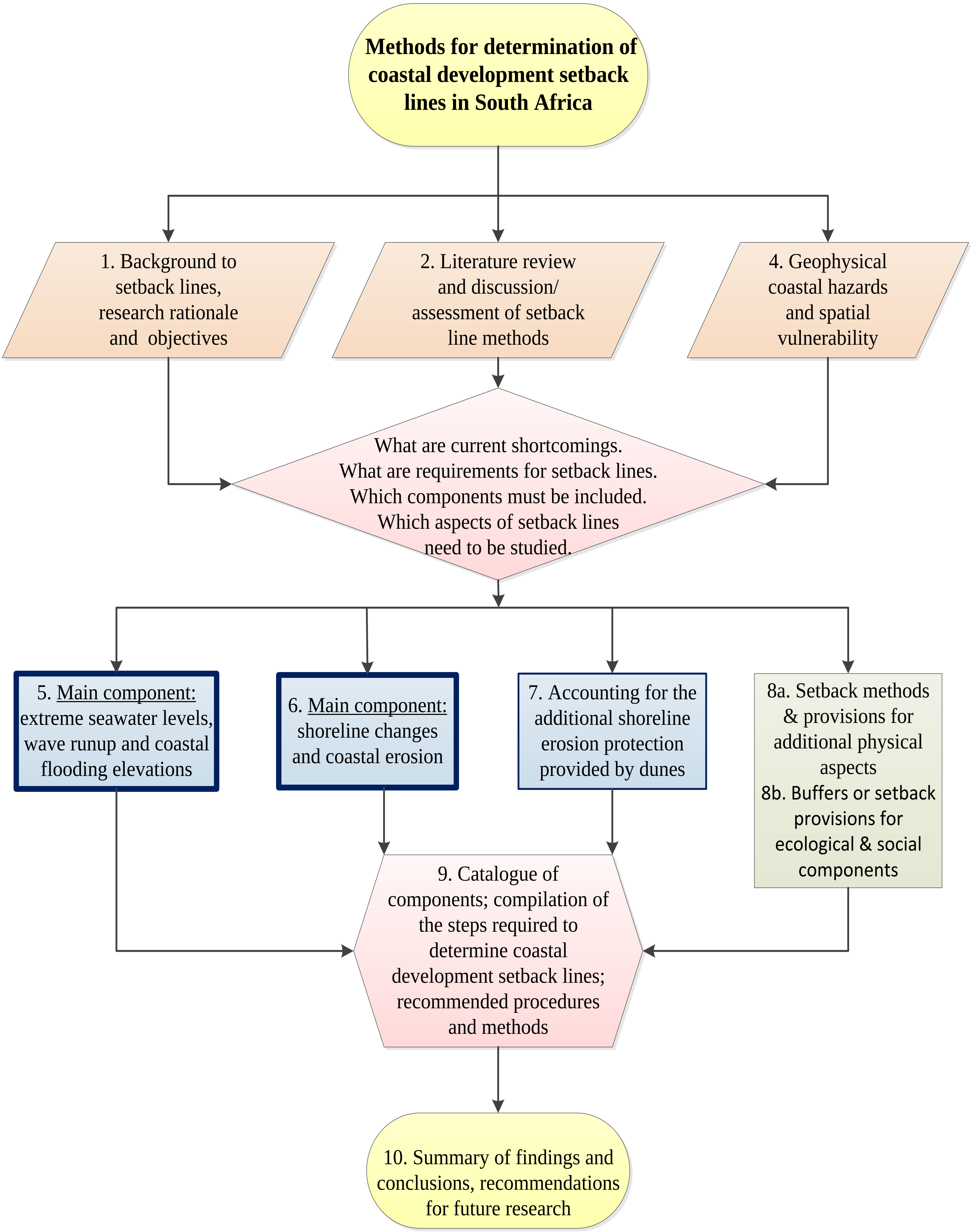


Figure 3.1: Path diagram (flowchart) indicating research methodology and thesis content

3.2. Data

The lists compiled here indicate most of the major types of data used for the research (mainly for testing & development of models and formulae, and application of methods in case study areas), as well as sources of most of the data. The data required for the research (mainly for model and formulae development & testing, as well as case study applications) included:

- Coastal topography and morphology (including dune and beach profile data);
- Bathymetry (including nearshore and inshore bathymetry);
- Sediment characteristics (especially grain size data);
- Present and future metocean climate (wave and wind regime, tides, atmospheric pressures, sea level, salinity, temperature, and future metocean scenarios);
- Historic shoreline (location) changes;
- Estuarine mouth dynamics and historic channel migration configurations;
- Coastal geography, geologic/geomorphology information.

Sources of data included:

- Offshore wave climate: NCEP hind cast wave data (NCEP 2013, from NOAA/NCEP WAVEWATCH III Model);
- Nearshore wave data: CSIR, TNPA (Transnet National Ports Association of South Africa);
- Aerial photographs, ortho-photographs: Surveyor General, CSIR;
- Topographic surveys: local authorities, CSIR;
- Remote sensing: Google Earth;
- Bathymetric data and charts: South African Navy Hydrographic office, CSIR, local authority;
- Tides, seawater levels: South African Navy Hydrographic office;
- Wind data: CSIR.

The primary data used and the sources were: wave and profile data collected by CSIR on behalf of TNPA, and coastal monitoring data (e.g. profiles, grain sizes, bathymetry) collated by CSIR on behalf of Ethekwini Municipality.

The specific data used for the different components of the research are discussed within the related chapters.

Chapter 4: Geophysical coastal hazards and spatial vulnerability

The purpose of investigating and quantifying coastal hazards and vulnerability as conducted in this chapter, is to provide a common understanding of what the primary physical (abiotic) hazards to coastal assets from the sea are, that could or should be addressed by setback lines. The research objective of this chapter is also to find or derive an appropriate methodology suited to South African conditions, by which the relative vulnerability of coastal areas can objectively be assessed and quantified, which in turn could be employed to prioritize areas where setback lines are most needed. Ultimately, the findings from this chapter also contribute towards ensuring that the setback line methods developed and recommended in the rest of this thesis do indeed address all of the relevant coastal hazards (and their drivers) as identified and discussed in this chapter.

4.1. Introduction

4.1.1. Relevance of coastal hazards, vulnerability and risk to coastal setback lines

Studying the hazards associated with coastal processes and dynamics, aids the planning and low-risk location of new development areas and infrastructure. The need, therefore, exists to determine areas of high risk or vulnerability (which includes prediction of future vulnerability under future climate change scenarios). Understanding the potential risk of physical impacts of coastal processes to both human and natural elements of the coastal zone facilitates the mapping of vulnerable areas. Following from the definition of a development setback line (Section 1.4), it is clear that the intention of the setback line is to protect amenities and infrastructure from "coastal hazards" and to reduce the risk of detrimental impacts to such developments by determining the location or line landward of which these fixed structures may be erected with reasonable safety. However, both the literature review (Chapter 2) and recent setback line workshops held in South Africa (e.g. by the University of Stellenbosch in October 2010 and October 2011, and the Western Cape Government on 11 October 2013), have highlighted the lack of consistent methods to determine setback lines as well as the major confusion around how to proceed. From these discussions, it can be concluded that the confusion and major differences, in part stem from not having a common understanding of which hazards the setback line could or should address and what are acceptable levels of risk of detrimental impacts occurring. In this context it is thus clear why the investigation and quantification of coastal hazards, vulnerability and risk is of relevance to coastal setback lines. In effect it can be said that the quantification of the space or area (location) over which selected or specific hazards are a risk, leads to the setback line.

Two recent and local examples of determining setback lines that incorporated (aspects of) a risk perspective, are those by the City of Cape Town (2012a, 212b) and Goble and Mackay (2013). However, as discussed in Section 2.3.2, there are significant problems with specific aspects of these two approaches.

4.1.2. Coastal hazards, drivers, vulnerability and risk factors

Coastal hazards and drivers

In the literature review (Section 2.4) it was concluded that the primary physical (abiotic) hazards to coastal infrastructure in South Africa from the sea are the following:

- Extreme inshore seawater levels resulting in flooding and inundation of low-lying areas;
- Direct (and indirect) wind and wave impacts;
- Coastal erosion and under-scouring of, for example, foundations and structures;
- A combination of extreme events, such as sea storms during high tides, will have the greatest impacts and will increasingly overwhelm existing infrastructure as climate change-related factors (e.g. SLR) set in.

The main metocean hazard drivers related to the above are thus waves and seawater levels (and to a lesser extent winds and currents).

Regarding wind hazards and the South African coastal zone, it is acknowledged that primary hazards to coastal infrastructure should include likely wind damage during high winds. The damage that may be done to infrastructure and housing by extreme winds should not be overlooked. However, extreme wind impacts may be felt far inland with no influence from the sea and, therefore, should rightfully be dealt with as a hazard to be included in risk assessment and response for virtually all areas, not specifically only the coast. Direct wind impacts therefore have no bearing on coastal setback lines, but indirect wind impacts can. The indirect wind impacts related to the coastal zone, manifest in areas where there is a high wind-blown sand transport potential that can impact on infrastructure, development or certain natural environments.

Tsunami hazards (mainly waves caused by earthquakes or undersea slope failures) and vulnerability are noted as not being considered in this research. Destructive waves and coastal flooding associated with tsunamis are considered to be a relatively low risk hazard for the South African coast (due to South Africa being located in a low seismic hazard area of the world, Figure 4.1). (Although this is beyond the scope of the present study, a focussed tsunami risk assessment for the South African coast could be conducted in the near future to properly assess vulnerability and quantify impacts or risks so that the need for tsunami-specific planning and adaptation can be ascertained.)

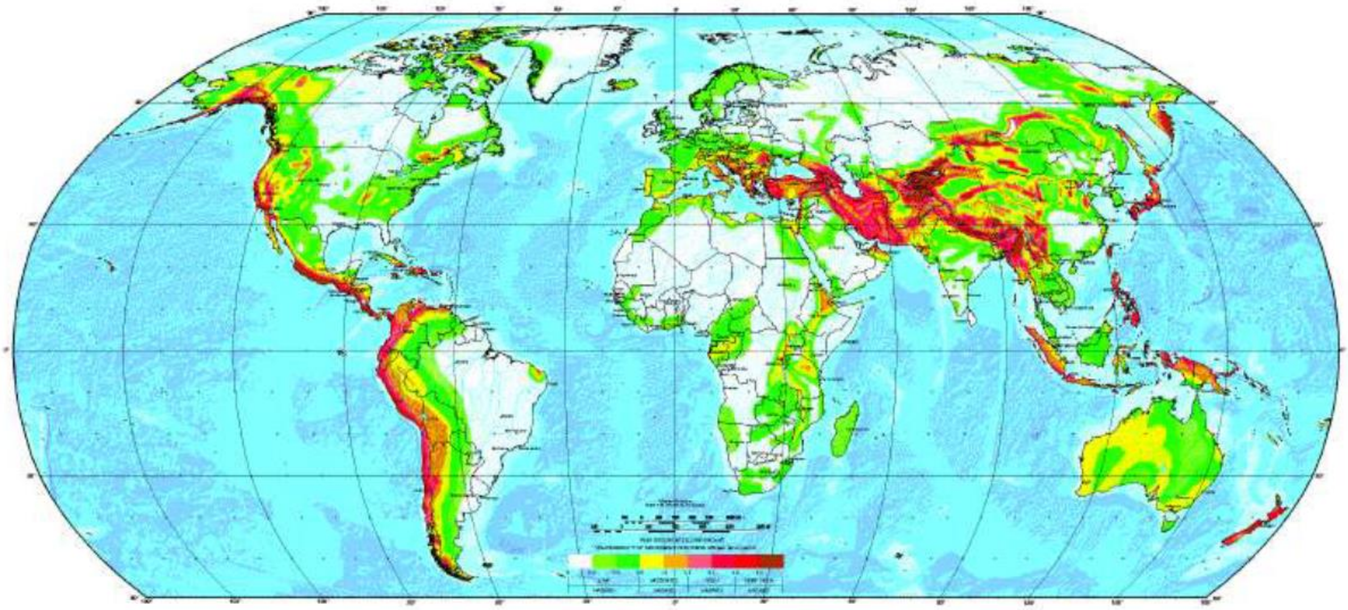


Figure 4.1: Global seismic hazard map *(Bosman, 2009)*

In also considering other abiotic “non-coastal/-marine” hazards and impacts in the wider coastal zone, one finds that there is value in noting the combined hazard of high seawater levels with flooding from rivers. It is well known that the heavy rains accompanying some sea storms also bring river floods that can be “backed up” in estuaries by high seawater levels along the coast (or due to high sand berms blocking the mouth). If such joint extreme events occur, they add to the destruction experienced by infrastructure and services. River and estuarine flooding studies need to take into account the possible effects of high seawater flooding levels, and attention must certainly be drawn to the potential combined flooding impact in the low-lying areas of cities or towns where rivers join the sea.

Coastal vulnerability and risk factors

Due to the diversity of the coastal characteristics, the hazards will have varying effects or impacts on the coastline. For example, even if a particular hazard, say wave height (or wave energy) was similar along some coastal areas, the different coastal characteristics e.g. erodibility (i.e. geologic characteristics or simply hard/soft nature, etc.) will affect shoreline stability differently (Figure 4.2).

According to Tinley (1985), about 80% of South Africa's more than 3000 km of coastline is 'soft' (erodible sand, or a mix of sand and rock) and that significant parts of the sand dune coast along South Africa are eroding.

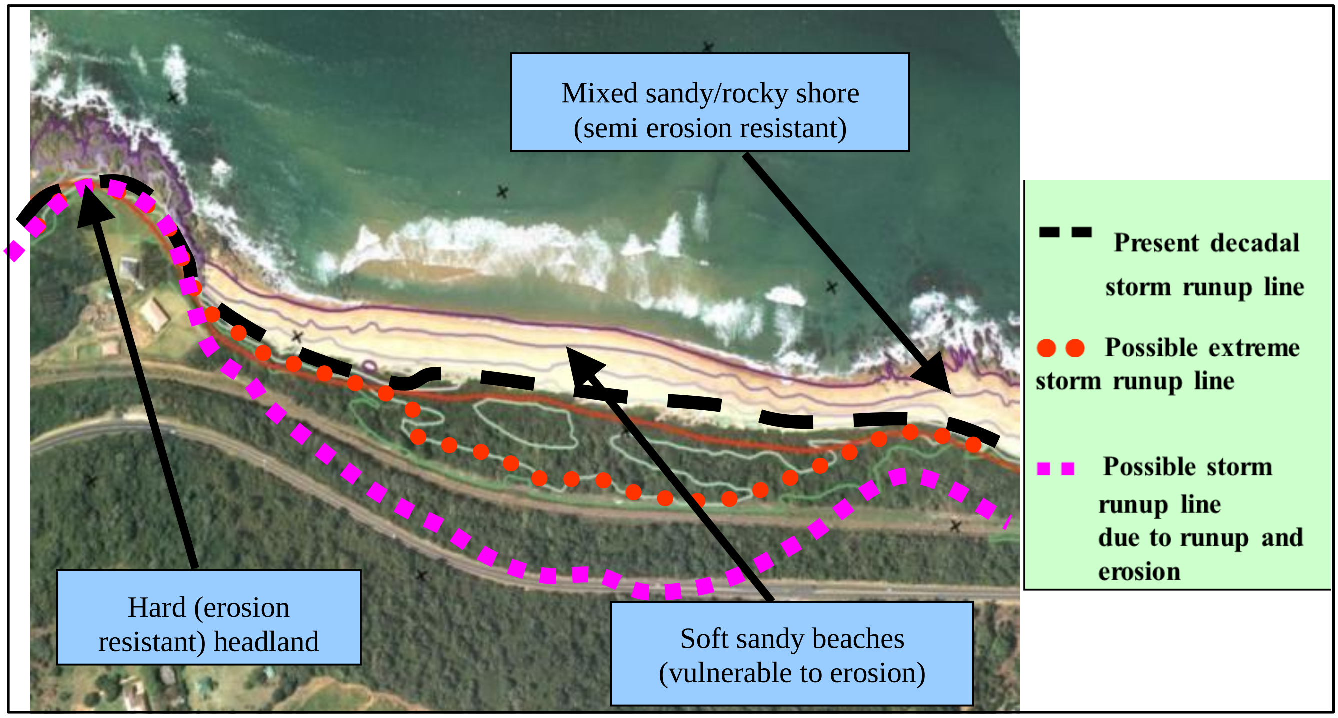


Figure 4.2: Varying coastal characteristics (e.g. erodibility) have varying effects on shoreline stability. *(Imagery from Ethekwini Municipality)*

A variety of local and regional factors affect the vulnerability of coastal areas to impacts from the sea such as erosion and flooding. For example, the southern part of the western Indian Ocean coastal zone is particularly vulnerable to such impacts due to it having:

- vast low-lying coastal plains including delta coasts (e.g. Beira);
- high population concentrations in close proximity to the sea;
- poverty and low capacity to defend infrastructure;
- susceptibility to cyclone activity;
- soft erodible coasts;
- inadequate and ageing existing coastal defences;
- direct exposure to high wave energy regimes in some parts;
- high reliance on goods, services and economic benefits provided by the coastal zone; and
- impacted natural coastal defences (e.g. dunes, mangroves, coral reefs).

Critical factors which strongly affect the vulnerability of coastal areas are: physical elevation, geology, distance of infrastructure to the sea (e.g. high water mark), exposure to storm waves and cyclones. Resilience is afforded by certain natural features (e.g. dunes, wetlands, mangroves, corals) and processes. Thus, the degree of impact by human activities on the natural system, and the integrity of the natural dune barrier (or other natural features) along the coastline, is of importance. How the impacts of abiotic coastal/marine processes will vary, depending on the diverse characteristics of the coastline (for example, hard/soft shores, steep/flat profiles, etc.), is discussed in detail in Chapter 8.

Coelho and Arende (2009) have defined risk as the product of vulnerability (likelihood) and consequence (in accordance with classical risk assessment methods). They provided a useful method and guideline to assess consequence in coastal areas from a holistic viewpoint that includes socio economic, ecologic and heritage parameters, as indicated in Table 4.1. So-called coastal risk assessments often actually do not go beyond an assessment of coastal hazards or vulnerability. Coelho and Arende (2009) complete the risk assessment by including an assessment of the consequence. Table 4.1 and the guidelines provided by Coelho and Arende (2009) also assist in making the final risk assessment less subjective.

Table 4.1: Consequence parameter classification** (Coelho and Arende, 2009) **for coastal areas.

Parameters	Very low 1	Low 2	Moderate 3	High 4	Very high 5
Population density (inhabitant/km^2)	< 500	≥ 500 < 1000	≥ 1000 < 2000	≥ 2000 < 4000	≥ 4000
Economical activity (involved persons)	0	1 to 10	11 to 30	30 to 50	> 50
Ecology	Zones without ecological relevance	Agricultural National Reserve	Ecological National Reserve	Zones of Ecological Protection	Natural Reserves
Historical heritage	Does not exist inheritance to preserve	There are some constructions not typical	Constructions and typical activities of a place	Historical regional constructions	Historical national monuments

The conceptual relationship between coastal hazards, vulnerability, setback lines and (residual) risk is depicted in Figure 4.3.

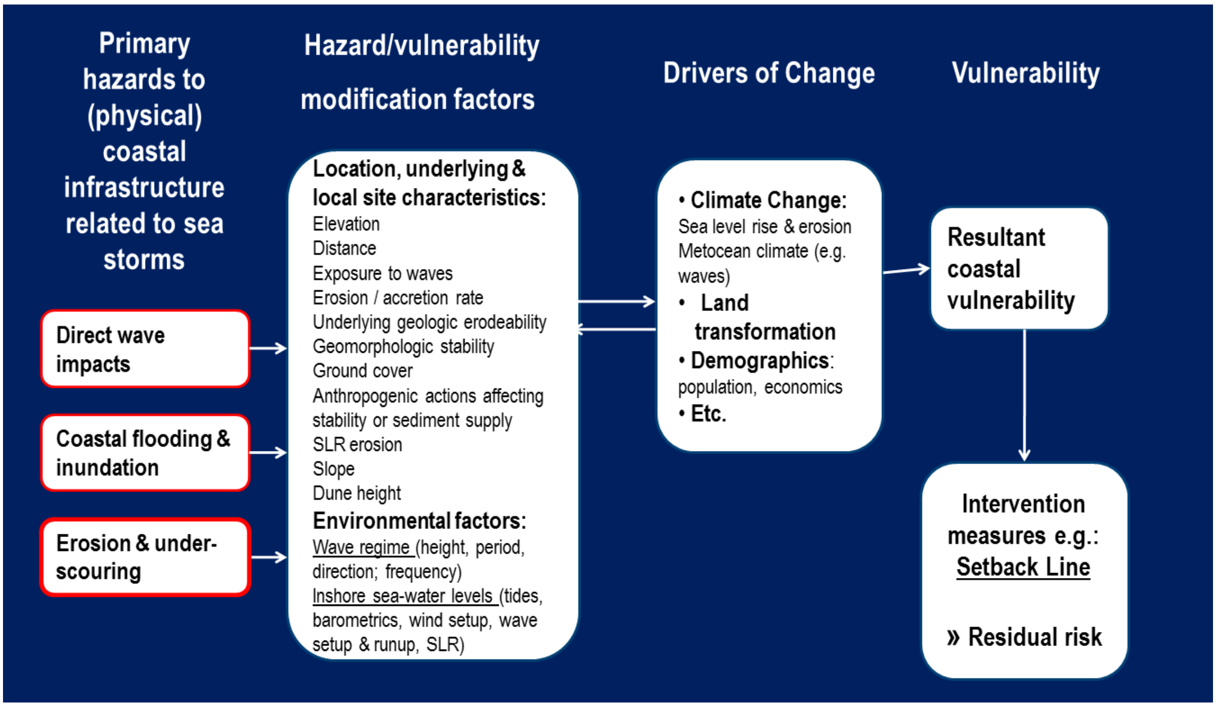


Figure 4.3: The conceptual relationship between coastal hazards, vulnerability, setback lines and (residual) risk.

4.2. Coastal hazard assessment method

4.2.1. Methods of assessing vulnerability of coastal areas and developments

Although DPIW (2009), DERM (2011) and especially Breetzke *et al* (2008), do not specifically contain vulnerability assessment methods, they do, however, contain information and guidelines on risks and response to coastal erosion that are particularly relevant to South Africa. The coastal vulnerability index (CVI) devised by the US Geological Survey and founded on six physical variables is found to be useful to assess the vulnerability of the coastline to climate change (Theiler and Hammar-Klose, 1999). These six variables are: geomorphology; coastal slope; relative sea level change; shoreline erosion/accretion rate; tidal range; and wave height. Dwarakish *et al* (2009) applied this same CVI method (as originally developed by Gornitz *et al*, 1997 and Thieler and Hammer-Klose, 1999) to the West Coast of India, while Pendleton *et al* (2010) applied it to the Gulf of Mexico. Another indicator, the coastal social vulnerability index (CoSVI) developed by Boruff *et al* (2005), is used to determine social-economic vulnerability of coastal areas to sea level rise (SLR). These indices can also be combined to give an overall vulnerability index, which is more informative and appears to be a viable approach to the South African situation. The methods of Dutrieux *et al* (2000) are considered to be more useful for integrated coastal zone management aimed at sustainability and protection or management of the natural environment, and are particularly useful for guidance on more detailed vulnerability mapping of smaller areas (e.g. islands).

The only directly related South African literature is that by Palmer *et al* (2011). Their proposed CVI method is based on seven parameters. The four cross-shore distance parameters are: beach width, dune width, distance to 20 m isobath, and distance of vegetation behind the back beach. (Palmer *et al*, 2011, do not clearly define how these distances are derived, stating that: “Data were captured … along transects between the low water mark and the back beach…. Beach width was calculated directly from transect length, while dune width was based on the width of the dunes behind the back beach coordinates. The distance to the 20 m isobath was calculated by identifying the nearest point of the 20 m isobath from the back beach coordinates.”) The fifth parameter accounts for the occurrence (%) of rock outcrops. The sixth parameter is an additional weighting for highly vulnerable sites, while the seventh is an additional weighting for estuarine areas. Palmer *et al*’s method is useful in that it can be applied based on remotely sensed data alone, and also because six of the parameters are considered to be relevant. However, the remotely sensed data needs to be of relatively high quality (thus potentially expensive). It also lacks some relevant parameters such as, for example, height or elevation, infrastructure location, and sufficient consideration of exposure to waves.

Appelquist (2012) presents a generic framework for assessing specifically inherent climate change hazards in coastal areas through a simple coastal classification and hazard evaluation system. It is interestingly presented as a graphical tool, the so-called “Coastal Hazard Wheel”, and is useful for application in developing counties in that it has limited data and computing requirements. However, it appears to be too coarse for use at sub-national scales (i.e. South African regions), does not include some important coastal vulnerability factors or parameters relevant to South Africa, and requires expert background knowledge and information about particular coastal types or environments some of which is less readily available. Rollason *et al* (2012) outline a methodology for applying the Australian Standard for Risk Management (AS/NZS ISO 31000:2009) to coastal zone management as prescribed within some Australian Government guidelines. Although the focus is more on managing risks and uncertainty related to coastal climate change effects or impacts, ideally, similar guidelines and standards could be drawn up for application in South Africa. However, significant development of appropriate standards would first be required in addition to collection and collation of suitable baseline coastal data.

The methods recently developed and applied in Portugal and Spain have a practical approach and are well-suited to the South African context (Theron *et al* 2010a). Jimenez *et al* (2009) have developed good coastal storm vulnerability assessment methods, but the input data requirements are considered to be too onerous for wide scale application in the African context. Jimenez (2008) provides a good description of how coastal vulnerabilities can be assessed for multiple hazards. However, from the literature describing the Portuguese and Spanish approaches, it was concluded that the set of parameters included in the method developed by Coelho *et al* (2006) and Coelho *et al* (2009), is pragmatic and most relevant for application to South African study areas.

4.2.2. Expansion and adaptation of a suitable vulnerability assessment method for South African study areas

The first part of the Coelho *et al* (2006) method is to assess the degree of exposure and vulnerability to coastal processes using the following nine indicators as the basis: foreshore elevation (e.g. ground level above MSL at seaward edge of infrastructure); distance (e.g. infrastructure) to shore; tidal range; offshore wave height; historical erosion/accretion rate; geology (type of rock or sediment); geomorphology (type: e.g. rocky cliff or river mouth); ground cover (e.g. forest, mangrove or urbanised/industrial); and anthropogenic actions (e.g. shoreline stabilisation intervention or sediment sources reduction). Specific limit values associated with each of the nine indicators are defined in the original method. Based on typical South African conditions and their local ranges, some of these values were adapted and the author selected appropriate ranges of values for each indicator, including for additional indicators as discussed below. Hazard/vulnerability assessment of a coastal area is done by obtaining the actual site specific values for each of the parameters and identifying within which indicator range these values lie. A vulnerability classification of Very Low (Vulnerability Score = 1) to Very High (Score = 5) can then be derived.

Three additional indicators have been identified here that are relevant to the South African study area, which have been added to the Coelho *et al* (2006) assessment methodology by the author:

- Degree of protection from prevailing wave energy (site location, coastline configuration or shape & orientation, bathymetry). Following a method proposed by Barwell (2011), scoring is done according to wave exposure as listed below and illustrated in Figure 4.4, in increasing order of exposure:
    - Leeside of large island or extensive spit on opposite side of incident waves (A);
    - Leeside of headland, rocky point or peninsula (A);
    - Partially sheltered from deep-sea wave energy (B);
    - Directly exposed to waves only slightly refracted from deep-sea (C); and
    - Directly exposed to storm wave attack, with narrow surf zone (D).

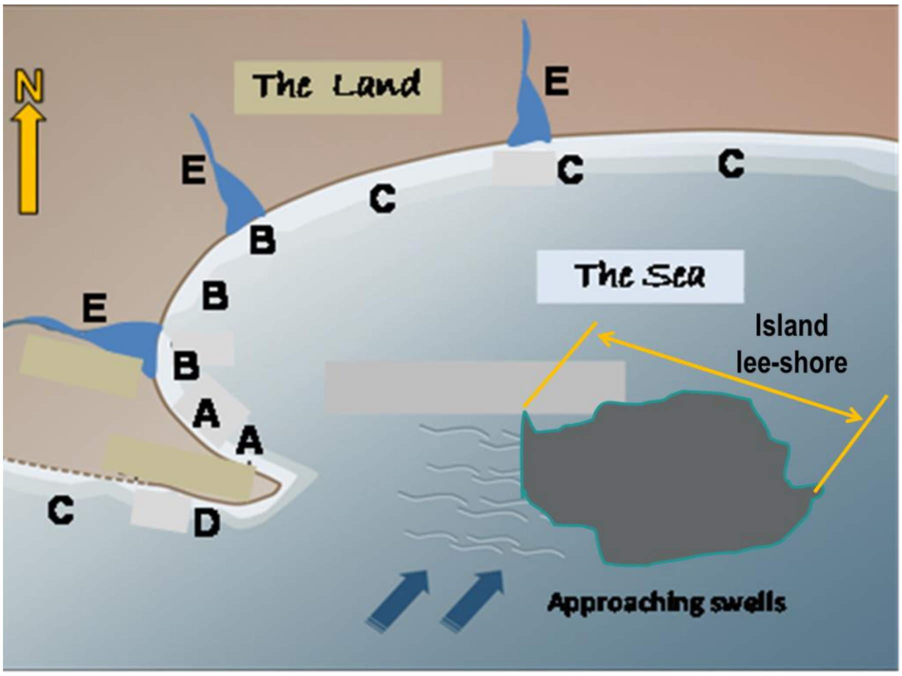


Figure 4.4: ***Degree of protection/exposure from prevailing wave energy (A – most protected, D – most exposed)***

(Wide areas of dense mangroves can also provide some wave protection, but this factor is already accounted for in the "ground cover" indicator mentioned before.) Additionally, if sites are located close to a river/estuary mouth (E), the vulnerability is scored more severely due to the risk of mouth meandering for example. This indicator therefore explicitly accounts for the differing vulnerability to incident storm waves due to location (and other wave modification factors), ranging from fully exposed open coast sites to well sheltered locations, for example within bays or on the leeside of headlands.

- Sea level rise erosion potential ("Bruun" factor in terms of inshore slope; see Section 8.2). Sea level rise is likely to result in flooding or inundation and coastal erosion. However, flooding/inundation vulnerability is already accounted for in the elevation and distance to shore. Thus, only the Bruun erosion potential needs to be assessed: for a specific amount of sea level rise, the erosion can be directly related to inshore slope. (Alternatively, the parameter to quantify could be taken as distance to the 10, 15 or 20 m depth contour; the choice depends on the "active" nearshore profile depth);

- Relative height (ideally volume) of the protective foredune buffer (i.e. the available sand reservoir). The importance of the foredune buffer as a natural coastal defence mechanism is discussed in Barwell (2011) and Chapter 7.

In tropical study areas (i.e. northern Kwazulu_Natal and Mozambique) two important additional indicators have been included by the author: cyclones (e.g. occurrence per annum in the vicinity of the study area); and protective corals or fringing reefs (alongshore extent as % of total shoreline length). About 2 cyclones per year enter the Mozambique channel (Figure 4.5).

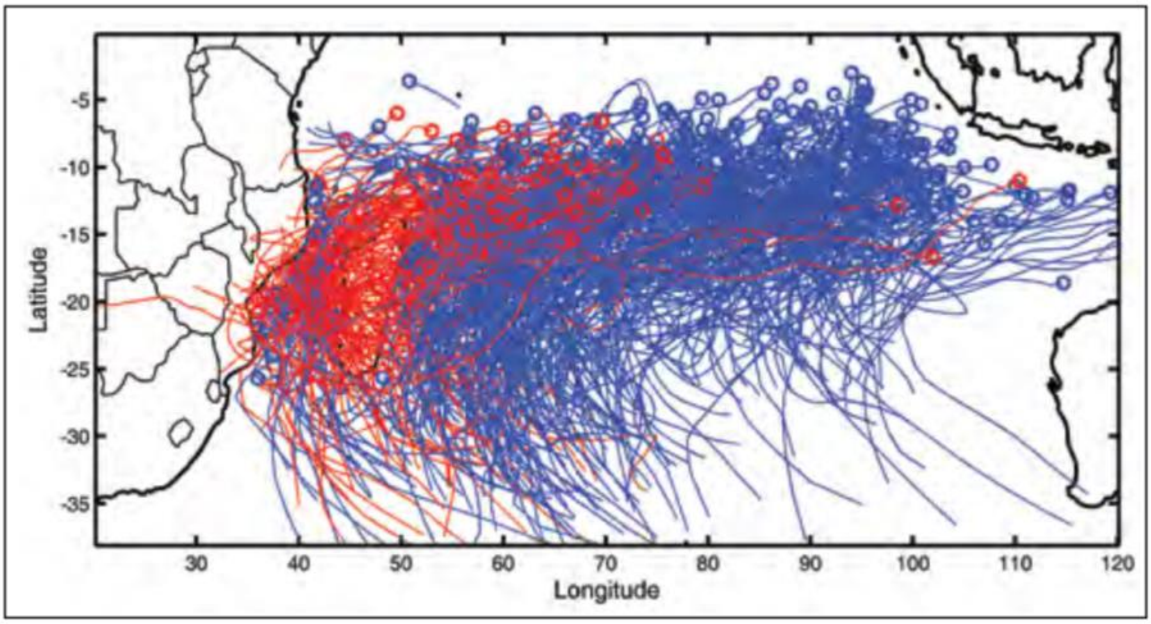


***Figure 4.5: Cyclone tracks November to April 1952 to 2007 in the SW Indian Ocean** (Land-falling cyclone tracks are red-marked. Mavume et al, 2009)*

As indicated in Figure 4.5, no cyclone has made land-fall on the South African coast since consistent tracking of cyclones in the region began in 1952. Also, very few cyclones approach farther south than Mozambique close to the South African coast (in the order of one per decade in northern KZN), and even when they do, they have usually lost much of their strength by the time that they enter South African waters. Thus, the potential for coastal impacts due to cyclones that are sufficiently significant to warrant additional assessment (separate from the effects of the wave related indicators already included in the selected vulnerability indicators), is extremely unlikely in the South African context and deemed to be inappropriate for virtually the entire South African coast. (Note, that other onland impacts resulting from extreme winds and especially riverine flooding due to distant cyclones have on vary rare occasions had severe consequences in KwaZulu-Natal province, such as caused by tropical cyclone Damoina in 1984.) Therefore, vulnerability to cyclones should rather not be included in assessments along the South African coast, other than potentially in northern KZN only, but should certainly be included in studies where this is more relevant, for example in Mozambique.

The protective function of corals or fringing reefs relates to their wave sheltering effect, which is strongly influenced by the crest level of the reef(s). Storm waves approaching the coast are affected by bottom topography, and shallow coral reefs that cause wave breaking, dissipate much of the incident wave energy - the higher the crest level, the more the wave energy is dissipated. In the South African context, coral reefs located in the inshore or nearshore zone only occur in limited areas along the KwaZulu-Natal coast. None of these reefs emerge higher than mean sea level, a few coral reefs have maximum crest elevations at about mean low water spring, while most have crest elevations of 5 m or more below MSL. Those few sites where significant areas of coral reefs with crest levels above -5 m to MSL occur, will at present experience significant wave sheltering during storm events. However, in the long-term, as the sea level rises due to climate change, existing topographic features including coral reefs will be located in deeper water and will have a reduced effect on waves approaching the coast. Areas landward of the reef will in time experience an amplified wave climate compared to the present. At low rates of eustatic sea level rise, healthy corals could potentially grow to match the rate of sea level rise, thereby retaining their protective effect, but recent findings seem to indicate faster rates of sea level rise (Section 5.2.4). Deeper water features including coral reefs may deepen to the extent that their effect on the wave energy impacting on the shoreline is negligible. In addition, the coral reef areas of Southern Africa and Mozambique in particular, are very vulnerable to climate change impacts, through coral bleaching (Obura, 2005). According to the IPCC (2013) there is medium confidence that coral reefs will be negatively affected by bleaching and by reduced calcification rates due to higher CO_2 levels (due to climate change). In other words, if the coast is subjected to the predicted sea-level rise, the protective role of the coral reefs will be diminished if their upward growth fails to keep pace (Theron and Rossouw, 2008). Thus, a possible or likely loss of coral due to climate change will also have additional detrimental impacts on the coast (such as erosion). Conservative planning horizons for coastal development should consider the long-term, which implies that the adverse effects discussed here should be taken into account. Areas landward of coral reefs currently “accustomed to” (or adapted to) significant wave shelter will in time experience an amplified wave climate compared to the present. For this reason the vulnerability indicator regarding coral reefs is set so that an increased alongshore extent of coral reefs is scored negatively (i.e. less corals is scored less vulnerable and more corals is scored more vulnerable).

Potential additional factors that could be considered in future studies are: characteristics of the winds (velocities above 12 km/h, that dominate during the dry season with an onshore component more than 20% of the time); pressures from human activities (to dunes and vegetation); and existing cross-shore beach width (e.g. to accommodate storm erosion or long-term recession trends).

Nevertheless, it is important to keep in mind which data is readily available to quantify a specific factor. “Double counting” must also be avoided, e.g. distance and elevation already account for slope on land; so, if distance and elevation are assessed, slope on land should not also be added as a factor. Seaward slope is, however, largely independent of on-land slope and is used specifically to assess vulnerability to erosion due to SLR.

In summary, a total of 14 vulnerability indicators have been determined as appropriate and applicable for the South African coast (also in terms of the available input data and information required). The 14 vulnerability indicators, the South African specific limit values associated with each of the indicators and the vulnerability classification ranges, are summarised in Table 4.2.

Table 4.2a: Vulnerability indicators, South African limit values for each indicator and vulnerability classification ranges applied for South African coastal vulnerability assessment.

#	Vulnerability Criteria	Vulnerability Classification & Score				
		Very Low (VL)	Low (L)	Medium (M)	High (H)	Very High (VH)
		1	2	3	4	5
1	TE: Topographic elevation (ground level above MSL at seaward edge of infrastructure or site in m)	>30	21 - 30	11 -20	6 -10	<5
2	DS: Distance to shore (infrastructure or site to MSL in m)	>1000	200 - 1000	50 -200	20 -50	<20
3	TR: Tidal range (m)	<1	1 - 2	2 – 4	4 – 6	>6
4	WH: Max. wave height (deep-sea, m)	<3	3 - 5	5 - 6	6 – 7	>7
5	EA: Erosion / accretion historical rate (m/yr)	>0 (accretion)	-1 to 0	-3 to -1	-5 to -3	< -5 (erosion)
6	GL: Geology	Hard rocks (Magmatic)	"Medium" hardness rocks (Metamorphic)	Soft rocks (Sedimentary)	Non-consolidated coarse sediment	Non-consolidated fine sediments
7	GM: Geomorphology	Mountains	Rocky cliffs	Erosive cliffs, Sheltered beaches	Exposed beaches, Flats	Dunes, river mouths, estuaries
8	GC: Ground cover	Forest/ Mangroves	Ground Vegetation, cultivated ground	Non-covered	Rural urbanised	Urbanised or industrial
9	AA: Anthropogenic actions	Shoreline stabilisation intervention	Intervention without sediment sources reduction	Intervention with sediment sources reduction	Without intervention or sediment sources reduction	Without intervention but with sediment sources reduction
10	Degree of protection from prevailing wave energy	Leeside of large island or extensive spit on opposite side of incident waves	Leeside of headland, rocky point or peninsula	Partially sheltered from deep-sea wave energy	Directly exposed to waves only slightly refracted from deep-sea	Directly exposed to storm wave attack, with narrow surf zone
11	Cyclones (occurrence/a)	0	>0 <1	1-2	>2-3	>3
12	Sea-level rise Bruun erosion potential (inshore slope)	<0.1 (1/10)	0.1– 0.029	0.03 – 0.014	0.015-0.005	>0.005
13	Corals/fringing reefs (alongshore extent as % of total length)	<10	10-30	30-50	50-80	>80
14	Relative height (m) of the protective foredune buffer	>20	10-20	5-10	0.5-5	<0.5

A futher definition or short description of each of the Vulnerability Criteria listed in Table 4.2a is provided in Table 4.2b so that the method is concisely described and to aid consistent implementation.

Table 4.2b: Definition of the vulnerability indicators listed in Table 4.2a.

#	Vulnerability Criteria	Definition or short description
1	TE: Topographic elevation.	Topographic elevation taken at ground level in m above MSL at the seaward edge of infrastructure (or site) being evaluated.
2	DS: Distance to shore.	Minimum horizontal distance in m between the shoreline (0 m MSL) and the infrastructure (or site) being evaluated.
3	TR: Tidal range.	Mean spring tidal range (m).
4	WH: "Maximum" wave height.	1 in 100 year deep-sea (> 200 m depth) significant wave height (m).
5	EA: Erosion / accretion historical rate.	Mean historical (medium to long-term) shoreline erosion / accretion rate (m/yr).
6	GL: Geology type.	Type of rock or sediment in terms of "hardness" or ability to "resist" erosion (the five classes are listed in Table 4.2a).
7	GM: Geomorphology classification.	Geomorphological classification in terms of typical shoreline variability or stability (the five classes are listed in Table 4.2a).
8	GC: Type of ground cover.	Type of ground cover (the five classes are listed in Table 4.2a).
9	AA: Anthropogenic actions.	Anthropogenic actions affecting shoreline stabilisation or reducing the source/supply of sediment to the site (the five classes are listed in Table 4.2a).
10	Degree of protection from prevailing wave energy.	This indicator explicitly accounts for the differing vulnerability to incident storm waves due to location (and other wave modification factors), as per the five classes listed in Table 4.2a (and explained at the hand of Figure 4.4).
11	Cyclone occurrence.	Number of cyclone occurrences/a in the study area.
12	Sea-level rise Bruun erosion potential.	The inshore slope between the shoreline (0 m MSL) and the seaward limit of the "active" nearshore profile (the profile close-out depth, which in South Africa mostly ranges between the 5 m and 20 m depth contour).
13	Corals/fringing reefs extent.	The alongshore extent of corals or fringing reefs as a % of the total shoreline length of the site.
14	Relative height (m) of the protective foredune buffer	The crest height (m) relative to HAT of the foredune buffer protecting the site from the sea (thus the dunes located between the sea and the infrastructure or location being evaluated).

If a vulnerability indicator is not applicable within the entire region in which an assessment is being made, then that indicator should rather be excluded from the assessment. For example, as discussed, cyclones do not occur along most of the South African coastal regions (such as in the Southern Cape case study discussed in Section 4.3). However, if a vulnerability indicator is not applicable in a certain part of the study area being evaluated, then it should still be included and scored according to the criteria provided in Tables 4.2a and 4.2b. For example, if a protective foredune buffer is not apparent in a certain part of the study area then the height in that location would be less than 0.5 m and the score awarded would be 5 (very high vulnerability),

Vulnerability indicators 1 and 2 (Table 4.2a) require the location of infrastructure or of the site being assessed. This means that in the case of an undeveloped strip of shoreline (without infrastructure) a decision first needs to be made about the location of the site being assessed before these indicators can be evaluated. If plans for development of a site do not (yet) exist (or it is not earmarked for development), then a logical and consistent assumption needs to be made about location of the site to be assessed. For example, the site may be taken as the area directly landward of the dune cordon (which area might be selected for development) or landward of some local zone restriction that might be applicable. Thus, the vulnerability of existing coastal infrastructure can be assessed, while the vulnerability of an undeveloped strip of shoreline can also be assessed, for example to determine suitability for future development (or regarding potential impacts to natural areas).

Almost all of the 14 indicators included in Table 4.2 can be assessed directly, based on the available input data. Some of the indicators require further interpretation or analysis of the input data to properly assess the vulnerability.

Erosion/accretion (# 5 in Table 4.2) is one of the most difficult indicators to quantify if historic data, such as aerial photography, is not available (as is for example the case for many areas of the Mozambican coast). Erosion (or accretion) can in such situations also be assessed from remote sensing (satellite images with change detection or analyses). Such image analysis procedures are affected by tides which make differentiating between ocean, beach and shallow water very difficult. Spatial resolution also plays a key role in the quality of the results, where for example, a ±30 metre accuracy achievable from free Landsat imagery is too inaccurate relative to typical changes found along the South African coast. It is concluded that high resolution satellite imagery or digital aerial photography or laser scanning (e.g. LiDAR) should be used to assess coastal variations/”stability”. To complement remote sensing techniques, use can be made of Google Earth images, and in-situ ground inspections. In all instances more emphasis is placed on the application of coastal engineering experience during observations and site inspections, rather than on typical coarse resolution “free” satellite imagery. If suitable pre 1980’s coastal aerial photography can be sourced, which should be the case for most South African locations, this can be very useful to quantify historic shoreline changes over a longer period. (Recommended procedures for quantifying the historic shoreline erosion/accretion rate are discussed in detail in Section 6.3. However, for completeness, the main steps of a basic acceptable procedure are very briefly listed here as follows: use aerial photographs and high resolution images that have been geo-referenced; plot/draw the historic high-water lines; plot the high-water line “distances from a reference point/line” versus “time” and fit a straight line through the data to obtain the long-term erosion/accretion trend.)

A conceptual description of a coastal hazard/risk "model" (based on the foregoing), which explains the functional relationships between components of the model, is presented in Figure 4.6. The "Coastal Hazard Assessment Model" approach could basically be described as an expert analysis of functional responses (ideally linked to process-based modelling).

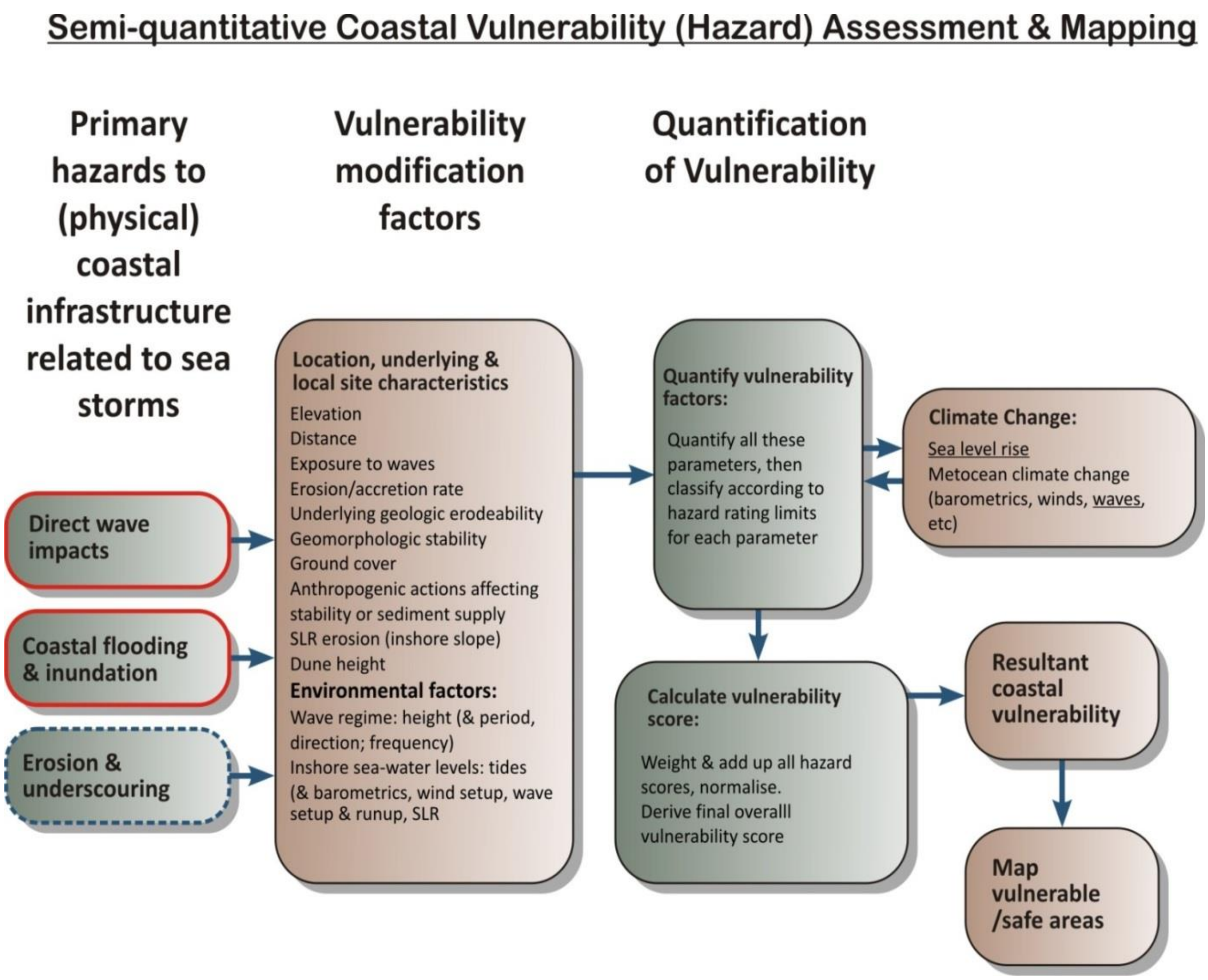


Figure 4.6: ***Conceptual description of the coastal hazard/risk evaluation model with functional relationships between components.***

Having developed a suitable assessment method to identify hazardous coastal areas, each particular hazard can then be investigated further to quantify the risk of occurrence or to determine which locations within an area are at risk from a specific event. The last step indicated in Figure 4.6 (i.e. mapping out vulnerable/safe coastal areas) would inform the final process of demarcating a coastal setback line. This final process (following this approach) includes defining a criterion for degree of (acceptable) vulnerability.

4.3. Vulnerability assessment case study

4.3.1. Application of the Coastal Hazard Assessment Method to Mossel Bay

Based on the coastal hazard/risk evaluation model as described in Section 4.2.2, a coastal vulnerability assessment was conducted of the Mossel Bay area (Cape St Blaize to Glentana). However, 2 of the 14 parameters/indicators were considered not to be applicable for the Mossel Bay study area (which would also not be applicable to most of the South African coast). These are the "cyclone" and "fringing (coral) reef" parameters, which would for example be applicable in Mozambique (and potentially the far northern part of KZN). Coastal points were defined along the Mossel Bay coast from Cape St Blaize to Glentana at 0.5 km intervals, as indicated in Figure 4.7. Coastal hazard/vulnerability assessments were conducted at each of these points.

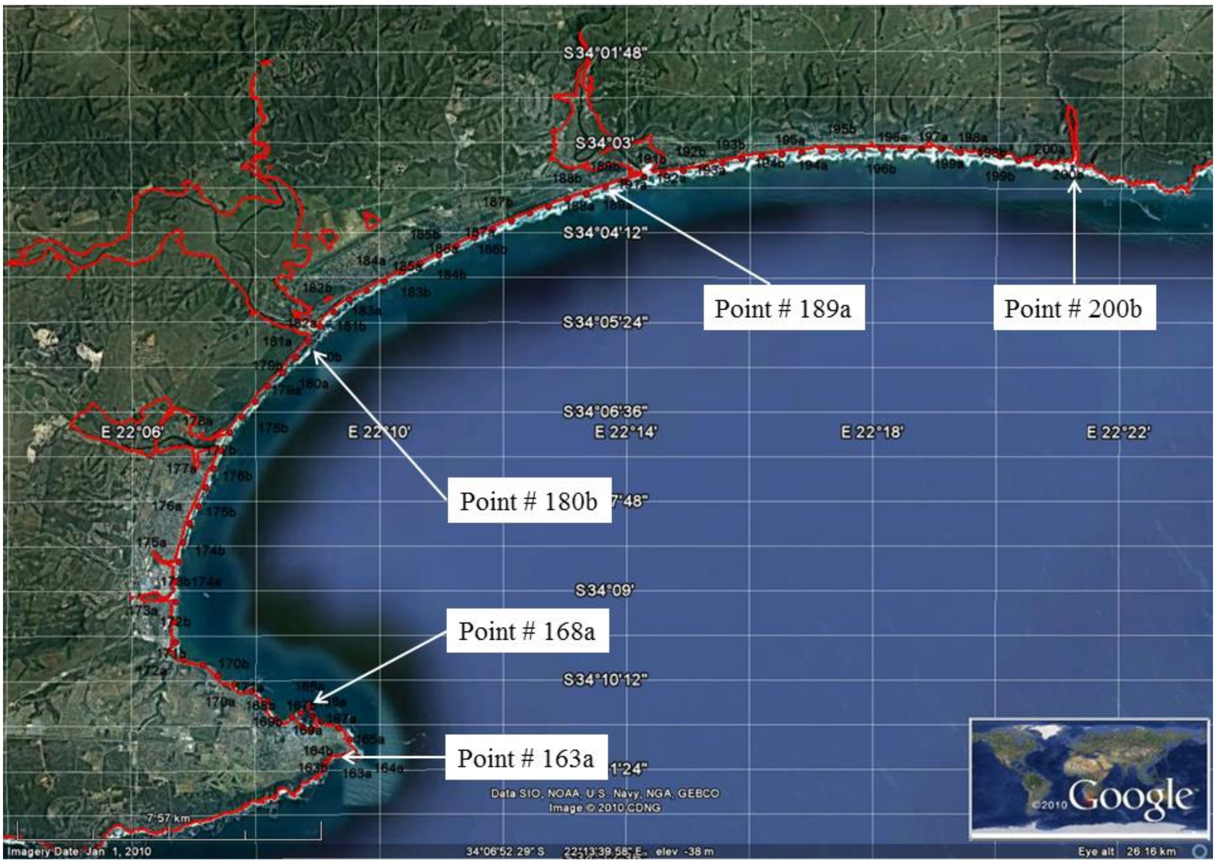


Figure 4.7: Mossel Bay Location of Coastal Points - 0.5 km intervals

Data was therefore obtained or derived for each of the remaining 12 parameters at each of the coastal points. These values were then scored according to the vulnerability classification for each parameter.

The individual scores were then added up and normalised to calculate the overall vulnerability score or rating for each coastal point. Appropriate weightings were also applied to the scoring to account for those parameters which have a greater influence on the vulnerability. An example of this process (for part of the Mossel Bay study area) is show in Table 4.3 (the point locations are indicated in Figure 4.7).

Table 4.3: Example of greater Mossel Bay coastal vulnerability assessment process.

#	Vulnerability Classification Indicator description	Area location: S of Cape St Blaize: **Point #**									**Location** M. Bay harbour : **Point #**	
		163a	163b	164a	164b	165a	165b	166a	166b	167a	167b	168a
1	**Elevation (m)**	1	4	4	4	4	4	5	5	5	5	5
2	**Distance (from infrastructure) to shore (m)**	2	4	2	3	4	4	5	5	5	5	5
3	Tidal range (m)	3	3	3	3	3	3	3	3	3	3	3
4a	Maximum offshore wave height (m)	5	5	5	5	5	5	5	5	5	5	5
5	**Degree of protection from prevailing wave energy (site location, coast configuration, bathymetry).** Scoring according to wave exposure as described in Table 4.2. Additionally, if located leeward of significant, surf zone/fringing reefs above low-tide (alongshore extent >50 % of total length), move down 1 class to next lower class.	5	5	5	5	5	5	5	5	5	1	1
6	Historic erosion / accretion rate (m/yr)	2	2	2	2	2	2	2	2	2	2	2
7	Geology	2	2	2	2	2	2	2	2	2	2	2
8	**Geomorphology**	2	2	2	2	2	2	2	2	2	2	2
9	Ground Cover	5	5	5	5	5	5	5	5	5	5	5
10	Anthropogenic Actions (Coelho & South African interpretation)	4	4	4	3	3	3	3	3	3	3	3
11	Sea-level rise erosion potential (Bruun; inshore slope)	5	5	5	5	5	5	5	5	5	5	5
12	Relative height (m) of the protective foredune buffer (i.e. the available sand reservoir).	1	1	1	3	3	3	3	3	3	3	3
Over-all	**Vulnerability rating - red double weights**	**M**	**H**	**H**	**H**	**H**	**H**	**H**	**H**	**H**	**H**	**H**
Over-all	**Vulnerability score - equal weights**	**3.1**	**3.5**	**3.3**	**3.5**	**3.6**	**3.6**	**3.8**	**3.8**	**3.8**	**3.4**	**3.4**
Over-all	**Vulnerability score - red double weights**	**2.9**	**3.6**	**3.3**	**3.5**	**3.6**	**3.6**	**3.9**	**3.9**	**3.9**	**3.4**	**3.4**

As indicated in Table 4.3, double weightings were also applied to the scoring to account for those parameters (marked in bold red) which have a greater influence on the vulnerability. This increased the range of overall vulnerability scores slightly (weighted scores ranging from 2.9 to 3.9 versus unweighted scores ranging from 3.1 to 3.8 – as per the last two rows in Table 4.3). Thus, the weighting increased the sensitivity of the vulnerability assessment, enabling a better relative comparison of the different coastal locations. In such instances, where certain parameters clearly have a greater influence on the vulnerability, appropriate weightings can therefore be applied to enhance the vulnerability assessment. Coelho *et al* (2006) have also illustrated the usefulness of broadly similar weighting approaches. However, it should be kept in mind that if area / region specific weightings are applied, the final vulnerability scores cannot readily be directly compared to scores from other areas. Thus, only if a consistent weighting scheme is applied throughout (such as applied in Table 4.3), is the wider general use of such weighting scheme recommended.

4.3.2. Mapping of detailed vulnerability assessment outputs

The vulnerability scores for 11 parameters at each coast point (representative of a 0.5 km section) along the study area, are summarised in the map depicted in Figure 4.8. (The SLR erosion potential indicator is not included on the map as it showed too little alongshore variation in this particular study area, due to the inshore bottom slopes (0 m to 20 m depth contour) being flatter than 1 in 400 at all the locations.) The vulnerability at each point is indicated by the colour code, ranging from blue “very low” (score in 0 to 1 band), to red “very high” (score in 4 to 5 band), as indicated by the legend.

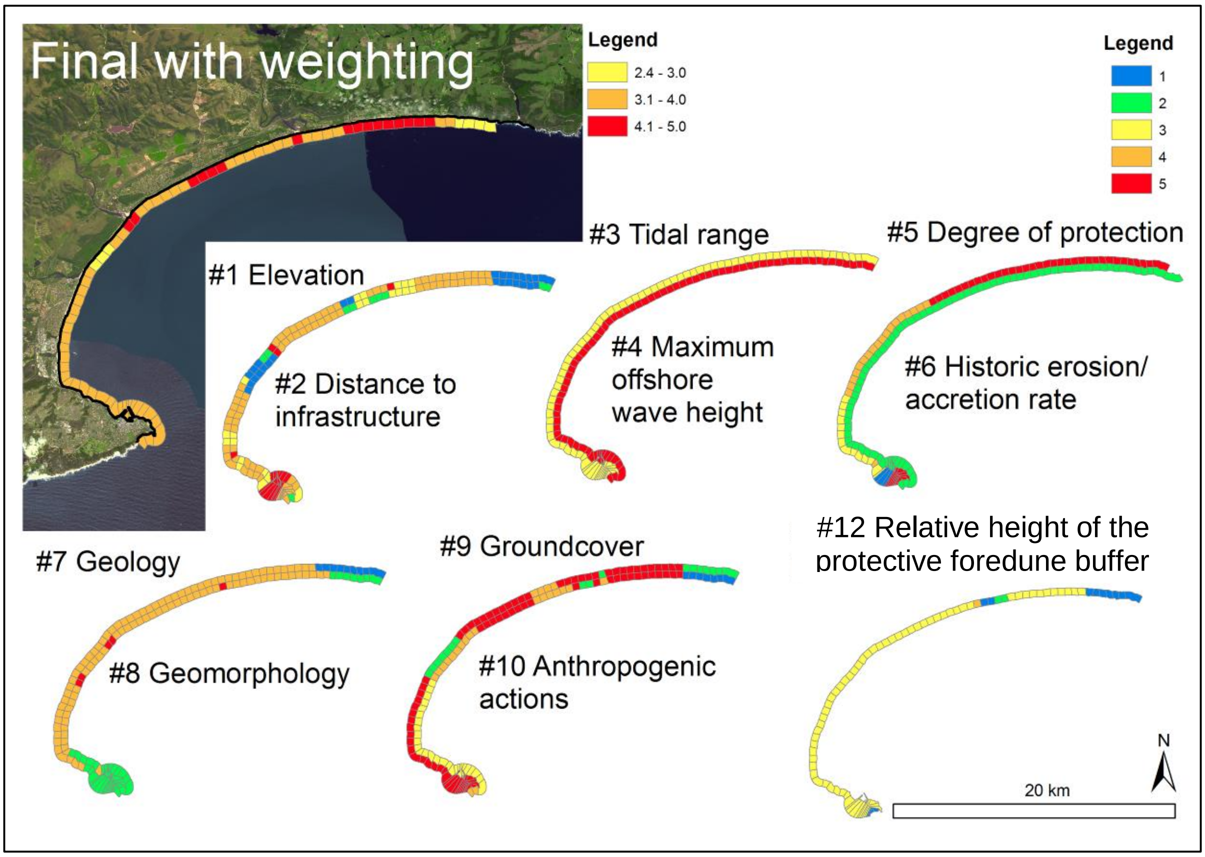


Figure 4.8: ***Mossel Bay vulnerability mapping showing 11 of 12 parameters. Vulnerability is measured on a scale of 1- 5 with 1= lowest vulnerability and 5 = highest vulnerability.*** *(GIS mapping by A Maherry)*

The total or overall vulnerability scores (all parameters combined) at each point (representative of a 0.5 km coastal section from Point # 163a to # 200a) along the study area, is summarised in the map depicted in Figure 4.9. The vulnerability at each point is again indicated by the colour code, ranging from blue "very low" (score in 0 to 1 band), to red "very high" (score in 4 to 5 band), as indicated by the legend. Besides the alongshore differences in vulnerability, it is interesting to note that all of the points are rated as having between medium to very high vulnerability.

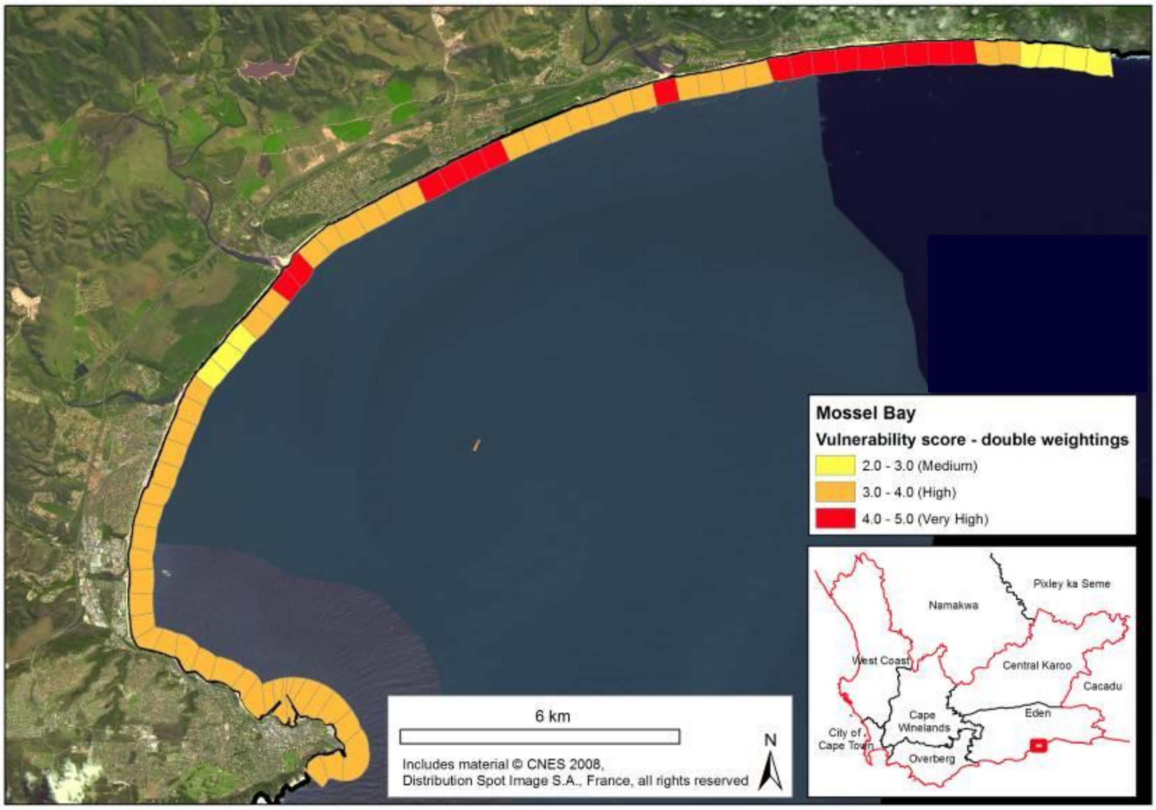


Figure 4.9: Mossel Bay vulnerability mapping – showing overall vulnerability rating when all 12 of the parameters in Table 4.3 are combined. *(GIS mapping by A Maherry)*

4.4. Conclusions

The primary physical (abiotic) hazards to coastal assets in South Africa from the sea are the following: extreme inshore seawater levels resulting in flooding and inundation of low-lying areas; direct (and indirect) wind and wave impacts; coastal erosion and under-scouring of, for example, foundations and structures; and combinations of extreme events, such as sea storms during high tides, which will have the greatest impacts, and will increasingly do so as climate change-related factors (e.g. SLR) set in. Thus, the most important drivers of risk to South African coastal infrastructure from erosion and coastal flooding, are waves, tides and future sea level rise. Critical factors which strongly affect the vulnerability of coastal areas are: physical elevation, geology, distance of infrastructure to the sea (e.g. high water mark), exposure to storm waves (and cyclones). Resilience is afforded by certain natural features (e.g. dunes, wetlands, mangroves, corals) and processes.

A review is presented of coastal hazard assessment methods, from which a practical evaluation technique, applied to European coastal conditions but applicable to South African conditions and data availability, was identified. This technique was adapted and further developed (building on methods proposed by Theron *et al*, 2010, 2012), to include additional forcing factors considered to be most relevant under South African conditions. In a case study, the coastal hazard assessment method was applied in Mossel Bay. Interpretation of the results enabled mapping of vulnerable areas to demonstrate the outcomes. The results were incorporated into a Geographic Information System (GIS) and mapped in order to spatially depict the results. An output of this study is thus a methodology for assessing coastal hazards and vulnerability. It is believed that this approach will be useful in assessing and mapping vulnerable coastal areas in South Africa, which in turn is a valuable contribution towards prioritizing areas where setback lines are most needed.

The case study illustrates that the vulnerability assessment method developed, is very suitable for identifying hazardous coastal areas, and to quantify the relative vulnerability of each location along the shoreline. Each particular hazard can then be investigated further to quantify the risk of occurrence or to determine which locations within an area are at risk from a specific event. Such results can then be interpreted or mapped by means of a GIS overlay procedure, to yield inputs into the demarcation of a comprehensive coastal processes setback line. Additional layers can easily be added, based on hazard and vulnerability assessment of ecological and social components (which are discussed in Section 8.6). These additional components typically require much wider consultation and public participation to resolve the issues, making such a GIS system particularly useful for this process. In this manner a hazard and vulnerability approach could be followed all the way through to a spatial quantification and mapping of areas at risk from specific coastal/marine hazards or impacts and thereby provide holistic inputs into the final demarcation of the setback line.

Such a “multi criteria” assessment is a more complete starting point for determination of setback lines than has been described in any of the literature reviewed for this thesis. The approach followed in the rest of this thesis and indeed found in the setback line literature, is to focus directly on the quantification of a smaller sub-set of critical parameters, such as historical shoreline location trends, storm erosion, flooding elevations, etc. However, the learning and findings from this chapter are taken into account by ensuring that the setback line methods developed and recommended in the rest of this thesis do address all of the relevant coastal hazards (and their drivers) as identified and discussed in this chapter. It appears that a worthwhile subject for future research would be to develop and test a complete “multi criteria” setback line approach based on further quantification of the comprehensive coastal vulnerability/risk method described here.

Chapter 5: Coastal flooding levels

The coastal flooding elevation (or level) is here defined as the highest point that the seawater can reach at the shoreline, due to the effects of natural events such as tides, winds and storm waves, which may be exacerbated in the long-term by processes such as sea level rise. To avoid impacts from coastal flooding, coastal developments, infrastructure and amenities therefore need to be located beyond this reach of the sea, thus at higher elevations located further landward. Determining coastal flooding levels and consequently where these points of highest reach of the sea lie on the land, is therefore one of the primary components of determining setback lines. Extreme seawater levels, storm surge and wave runup predictions are all part of determining coastal flooding elevations, which is one of the two major abiotic components of setback lines. Thus, the research objectives of this chapter are to: find or derive appropriate methods to determine coastal flooding levels in a "data poor" environment, that can be efficiently applied in large study areas, but that are still sufficiently robust and defendable; and to make recommendations for appropriate, practical and implementable methodologies to determine the coastal flooding components of setback lines in South Africa.

5.1. Introduction and approach

As indicated by the literature review (Section 2.4.1) and confirmed by the findings of Chapter 4, the most significant drivers of deleterious abiotic impacts on the South African coast (of natural causes) are usually sea storms (i.e. high waves and mostly to a lesser degree high winds) combined with high seawater levels resulting in coastal flooding. This chapter is therefore focused on the quantification of the components of high inshore seawater levels and the specific combinations thereof that determine coastal flooding elevations. Analyses of data records and numerical models are applied to the specific drivers that have significant effect on coastal flooding elevations. To avoid confusion or misinterpretation, it is important to define here what is meant by the term "coastal flooding" level or elevation. In this thesis it is defined as the highest point that the seawater can reach at the shoreline, due to the effects of natural *events* such as tides, winds and storm waves, which may be exacerbated in the long-term by processes such as sea level rise. The implication in *not* that areas located within the coastal zone below this level would permanently (or for extended periods of several days or longer) be inundated ("flooded") by seawater. During extreme events such as sea storms (resulting in surge and/or wave runup), thus for relatively short periods ranging in the order from typically seconds to hours, the seawater may reach up to a certain elevation on the shoreline, which is here called the coastal flooding level or elevation.

From the literature review (Section 2.5) it is concluded that the drivers or components of extreme inshore seawater levels most significant to the South African context are the tides (South African spring tides are about 1 m above MSL but reach up to + 3.7 m above MSL in Mozambique), potential SLR, wave setup and wave runup. Various combinations of the first three effects (i.e. tides, potential SLR and wave setup), as well as wind and barometric setup, can give rise to extreme “still-water levels” of the sea near the shoreline, commonly referred to as storm surge. A concise definition of storm surge can be expressed as follows: “an abnormal rise of the *mean* seawater level generated by a storm and/or a meteorological event, over and above the astronomical spring high tides.”

As noted before, the above drivers of storm surge (extreme inshore “still-water” levels) *exclude* the added effect of wave runup, which can reach even higher elevations. Wave runup is the rush of water up the beach slope beyond the still-water level (i.e. the swash zone). Extreme wave runup elevations (which include components of storm surge) are a good indicator of extreme coastal flooding elevations. (Extreme wave runup elevations have a much shorter duration than storm surge effects and are maintained for only relatively short periods, typically from a few seconds to less than a few minutes.) A definition sketch of the various components leading to extreme inshore seawater levels (identifying the components of tide, barometric/hydrostatic setup, wind setup, wave setup, wave runup and SLR) is presented in Figure 5.1.

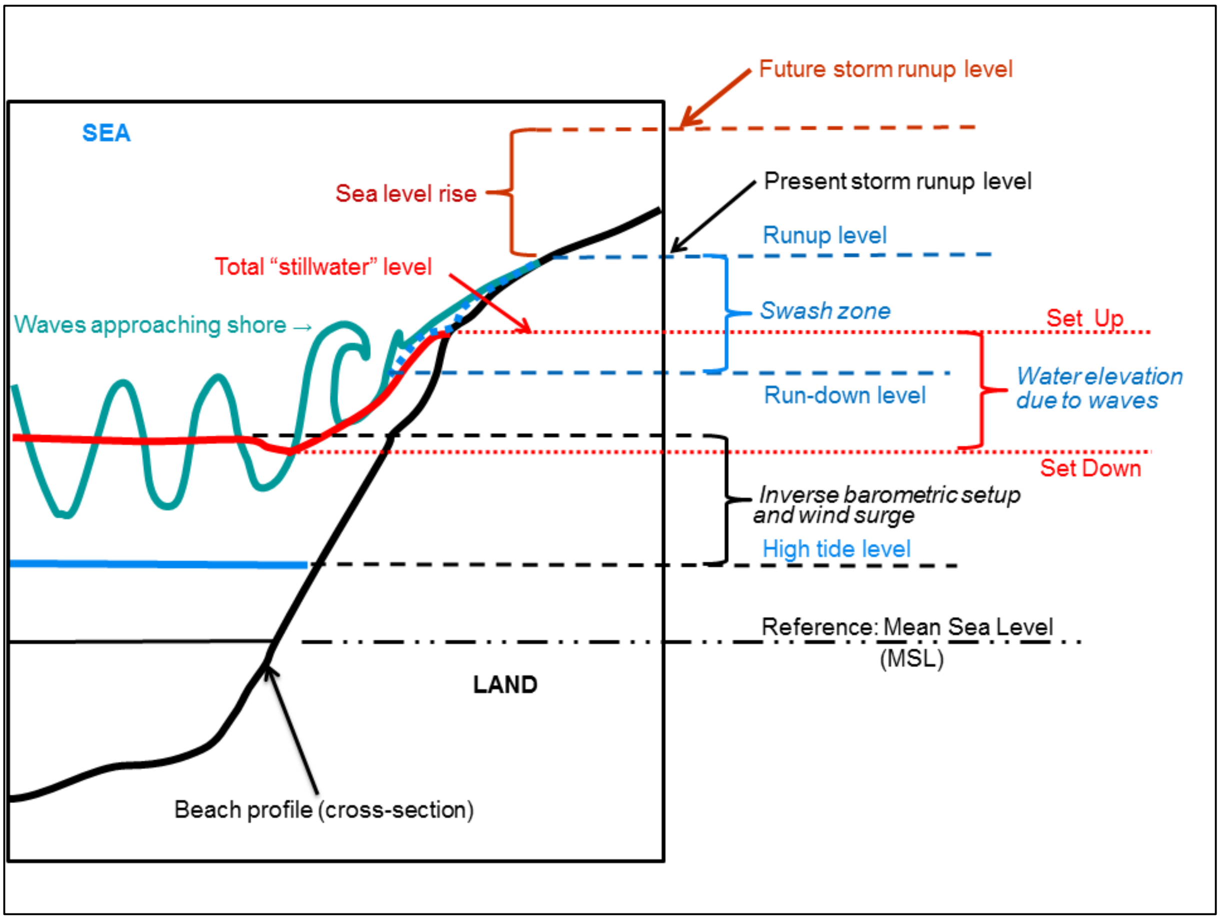


Figure 5.1: Definition sketch of the various components leading to extreme inshore seawater levels *(adapted from Theron et al, 2012)*

5.2. Prediction of high inshore seawater levels (storm surge)

5.2.1. South African inshore sea level recordings

South African sea level data and analyses

The South African Navy Hydrographic Office (SANHO) is responsible for the tide gauges in the principal harbours of South Africa, as well as the dissemination of the Annual Tide Tables (SANHO, 2012). While the monitoring and forecasting of expected tidal conditions are important for maritime safety, these are also important for a full understanding of coastal processes including extreme inshore seawater levels. The SANHO Sea Level Network, operating at 10 harbours along the South African coast, is shown in Figure 5.2, with sea level stations located at Durban, Port Elizabeth, Simon's Town, Port Nolloth, Saldanha Bay, Cape Town, Mossel Bay, Knysna, East London and Richards Bay.

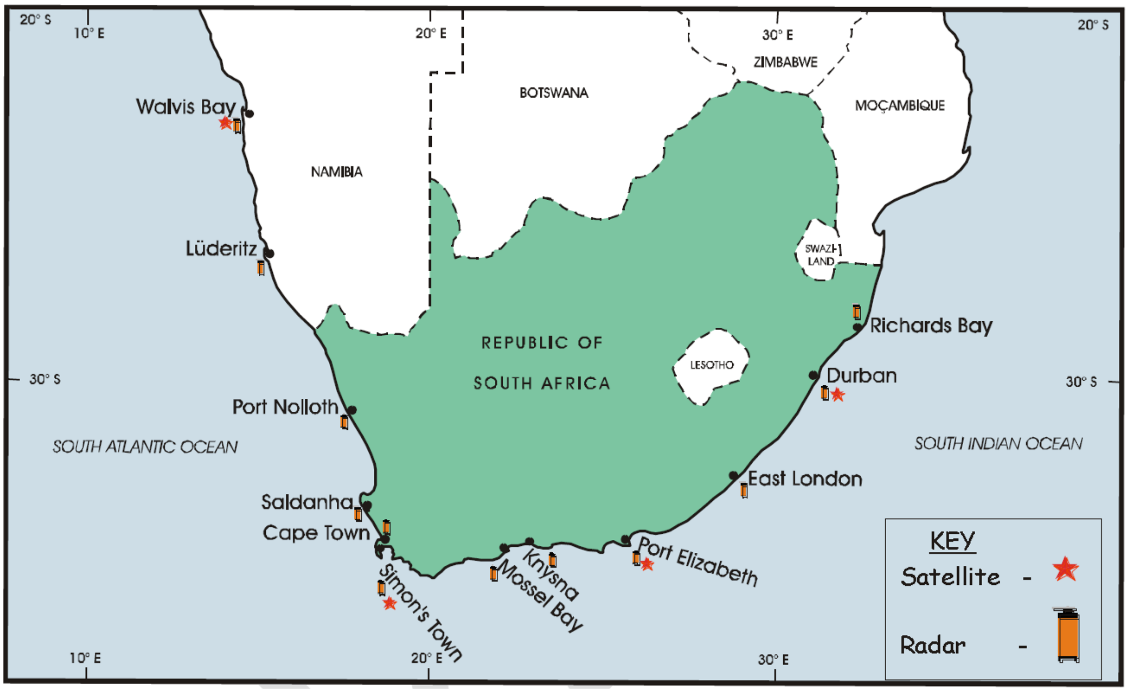


Figure 5.2: South African Sea Level Network *(Figure courtesy of M Rossouw, pers com)*

Tides around the South African coast are remarkably uniform (e.g. Wijnberg, 1993), being strongly semi-diurnal in character and having a tidal range of about 2 m, which lies on the border between micro and mesotidal coastal environments. Spring high tides occur within 50 minutes at all stations around the coast, thereby providing generally weak tidal currents (Searson & Brundrit, 1995). The tidal range for the South African coast is summarised in Table 5.1. (In comparison, spring tides reach up to about 3.7 m above MSL in Mozambique.) Tidal fluctuations include the effects of long-period (4.4 years and 18.6 years) constituents of a few centimetres, which determine the highest astronomical tides (HATs) at each location, as included in the table.

Table 5.1: Summary of tidal levels around the South African coast *(SANHO, 2012)*

Location	MHWS (m to MSL)	HAT (m to MSL)
Port Nolloth	0.99	1.49
Saldanha Bay	0.89	1.17
Cape Town	0.92	1.20
Simon’s Town	0.95	1.25
Hermanus	0.99	1.28
Mossel Bay	1.17	1.51
Knysna	1.12	1.42
Port Elizabeth	1.02	1.28
East London	1.10	1.36
Durban	1.10	1.39
Richards Bay	1.10	1.46

The highest spring tides of the year are equinoctial, occurring in spring and autumn. The highest equinoctial spring high tides, close to the level of the HAT, occur every 4.4 years (recently in 2007, 2011 and 2015). HAT is the highest predicted astronomical tide under average meteorological conditions over a full 18.6-year nodal cycle and is not reached every year. The various high tides, as summarized in Table 5.1 for the South African coast, represent one of the significant components of extreme inshore sea levels.

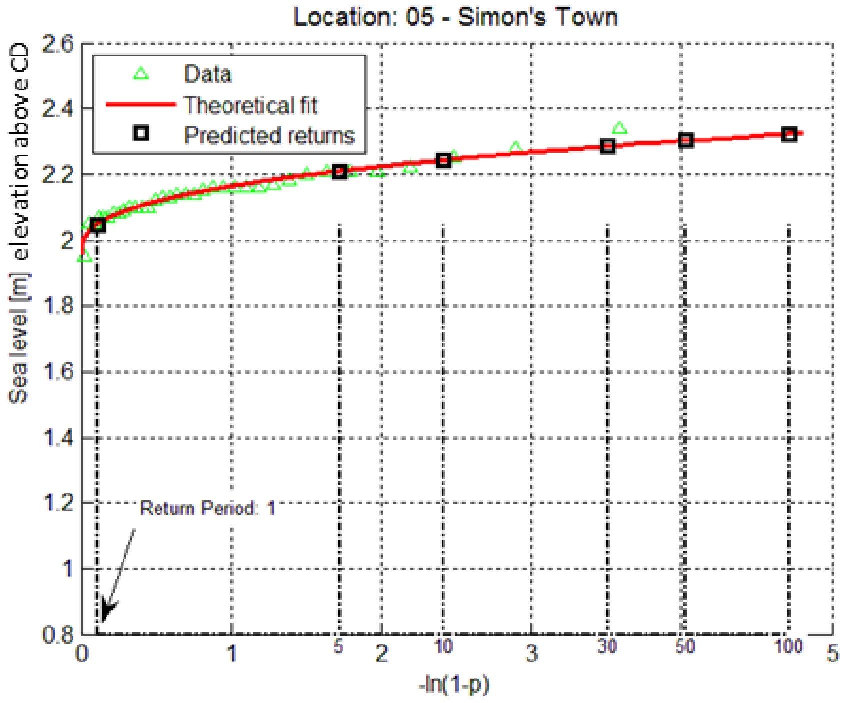
The difference (both up and down) between sea level recordings and tidal predictions is mainly due to atmospheric forcing of the sea level through the variation in atmospheric pressure, wind effects (dependent on strength, duration, fetch and direction), and shelf waves (Wijnberg, 1993). In the South African context, wind setup and tropical cyclones only affect highly localized areas, while other phenomena such as long period waves (e.g. edge waves) and tsunamis are only of secondary importance (Wijnberg, 1993). (Edge waves or bound infragravity/long waves with wave periods ranging from 25 to 350 seconds, are associated with the groupiness of swell and are related to the radiation stress of the waves (van Dongeren *et al*, 2002). Additional sources of long waves can be tsunamis and low-pressure atmospheric systems (de Jong *et al*, 2002).) A suitable tidal filter can be applied to hourly sea level measurements to provide a series of daily mean sea levels (Doodson & Warburg, 1941). The water level residual (i.e. the difference between the mean sea level and the actual water level minus the tidal effect) has a range typically varying from 70 cm on the west coast to over 90 cm on the south coast and propagates around the coast (related to the main weather systems, including atmospheric forcing, wind effects and shelf waves) mostly from west to east (Van Ballegooyen, 1996 and Schumann & Brink, 1990). The Southern Cape coast is the most sensitive to shelf wave activity which is considered to be related to the significantly wider continental shelf in this region (Wijnberg, 1993).

Brundrit (2009) updated observations and predictions for Simon's Town from Searson and Brundrit (1995). According to Brundrit (2009) the 1-in-1 year level is 1.28 m (MSL), and therefore extreme sea levels exceeding the HAT can be expected on an annual basis. HAT should be expected to be exceeded at least every 2.5-3 years all around the South African coast (Wijnberg, 1993). The maximum sea level actually observed over a 53-year period is 1.52 m (MSL) (Brundrit, 2009).

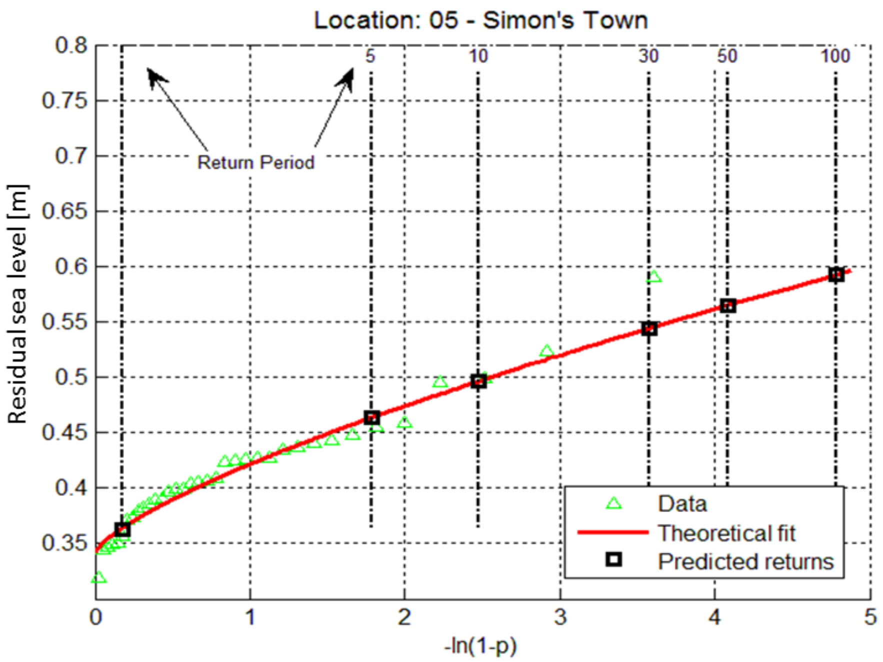
Analyses of extreme sea level recordings

To determine sea level extremes that are not associated with tidal changes, a tidal filter is applied to the recorded sea level data. Thus, most of the water level variations with periods shorter than say 40 hours can be removed from the recorded sea level data. A ratified South African sea level data set is available from the Sea Level Center of the University of Hawaii. This data set covers a period of about 45 years (1965–2010), but contains substantial gaps in some instances. For example, the presently available data for all measuring locations in South Africa ranges from about 80% complete over 50 years (the best - Simons Town) to about 40% complete over 40 years (the worst - Richards Bay). The incompleteness of the data from some stations detracts from the accuracy of the predictions at those locations, but the extreme results presented here can nevertheless be described as very good estimates.

In this manner, an extreme analysis was recently conducted on the residual water levels after the astronomical tide component was removed (Theron *et al*, 2014). (In this study, a three-parameter Weibull distribution was then fitted to the annual maxima of the residual data, in other words by selecting the maximum value on an annual basis.) Taking Simons Town as an example, Figure 5.3(a) presents the best fit for the Weibull distribution on the data set which still includes tides, while the fit for the residual (after removal of the tidal signal) is shown in Figure 5.3(b).



(a)



(b)

Figure 5.3: (a) Extreme estimate fit for sea water levels including tides, with respect to chart datum, and (b) Extreme estimate fit for residual (excluding tides) water levels *(Figure courtesy of C Rautenbach, pers com.)*

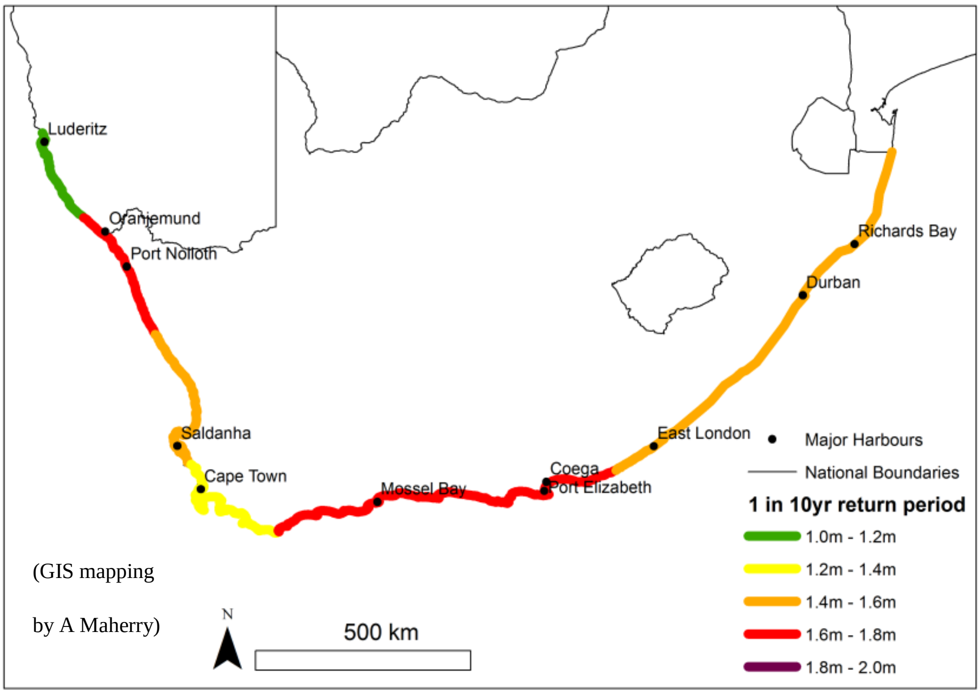
The results for all of the South African stations for extreme seawater levels excluding tides (i.e. residuals) and various return periods are summarized in Table 5.2 (a) above CD and (b) above mean sea level. (The water level residual by definition excludes the tides, and does not need to be referenced to a datum level, as it is a difference or magnitude and not an elevation. However, in this case, the water level data was analyzed relative to CD, and therefore Table 5.2 (a) indicates the residuals above CD due to the offset between CD and MSL being included in the data. The values in Table 5.2(b) are equivalent to the pure residuals excluding the offset between CD and MSL.)

Table 5.2: Extreme residual still-water level estimates *(from Theron et al, 2014)*

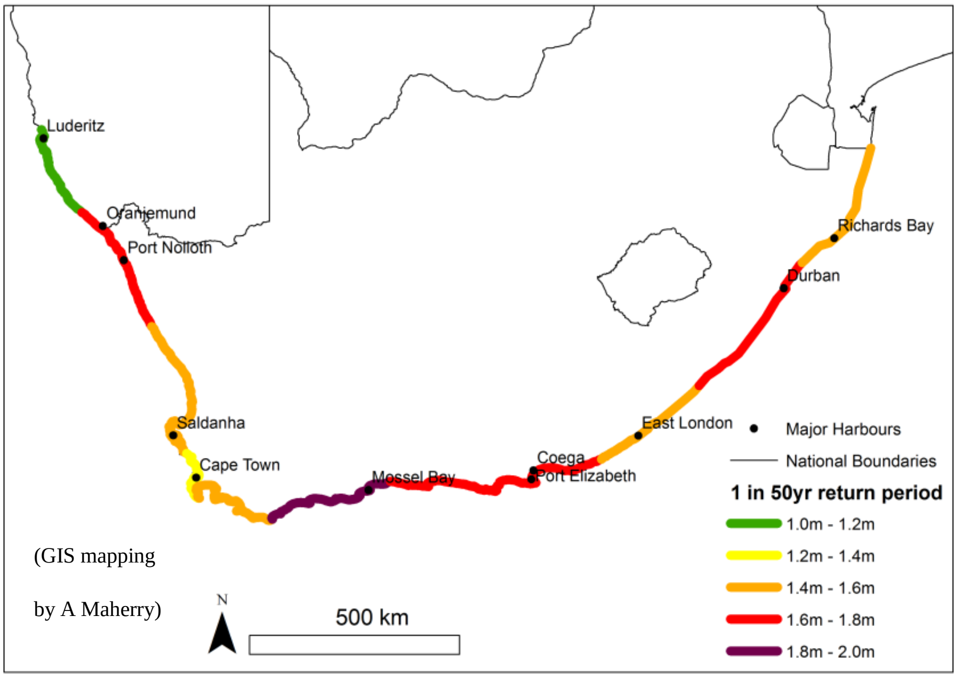
(a) Above CD	Return period in years							
	1	5	10	25	30	40	50	100
Saldanha Bay								
Residual sea level to CD (m)	1.25	1.38	1.40	1.43	1.43	1.44	1.44	1.46
Cape Town								
Residual sea level to CD (m)	1.21	1.34	1.36	1.38	1.38	1.39	1.39	1.40
Simon’s Town								
Residual sea level to CD (m)	1.21	1.31	1.34	1.38	1.39	1.4	1.41	1.44
Mossel Bay								
Residual sea level to CD (m)	1.49	1.70	1.76	1.82	1.83	1.85	1.86	1.90
Knysna								
Residual sea level to CD (m)	1.44	1.64	1.68	1.72	1.73	1.74	1.75	1.77
Port Elizabeth								
Residual sea level to CD (m)	1.4	1.63	1.66	1.69	1.7	1.70	1.71	1.73
East London								
Residual sea level to CD (m)	1.29	1.42	1.46	1.51	1.52	1.54	1.55	1.58
Durban								
Residual sea level to CD (m)	1.33	1.51	1.56	1.63	1.64	1.66	1.67	1.72
Richards Bay								
Residual sea level to CD (m)	1.34	1.52	1.55	1.58	1.59	1.59	1.6	1.62

(b) Above mean sea level	Return period in years							
	1	5	10	25	30	40	50	100
Saldanha Bay								
Residual sea level (m)	0.38	0.52	0.54	0.56	0.56	0.57	0.58	0.59
Cape Town								
Residual sea level (m)	0.38	0.51	0.53	0.55	0.56	0.56	0.57	0.58
Simon's Town								
Residual sea level (m)	0.36	0.46	0.5	0.54	0.54	0.56	0.57	0.59
Mossel Bay								
Residual sea level (m)	0.55	0.77	0.83	0.89	0.9	0.92	0.93	0.97
Knysna								
Residual sea level (m)	0.65	0.86	0.9	0.94	0.94	0.95	0.96	0.99
Port Elizabeth								
Residual sea level (m)	0.56	0.8	0.83	0.86	0.86	0.87	0.88	0.89
East London								
Residual sea level (m)	0.57	0.71	0.75	0.8	0.81	0.82	0.83	0.86
Durban								
Residual sea level (m)	0.41	0.59	0.65	0.71	0.73	0.75	0.76	0.80
Richards Bay								
Residual sea level (m)	0.32	0.51	0.53	0.56	0.57	0.58	0.58	0.60

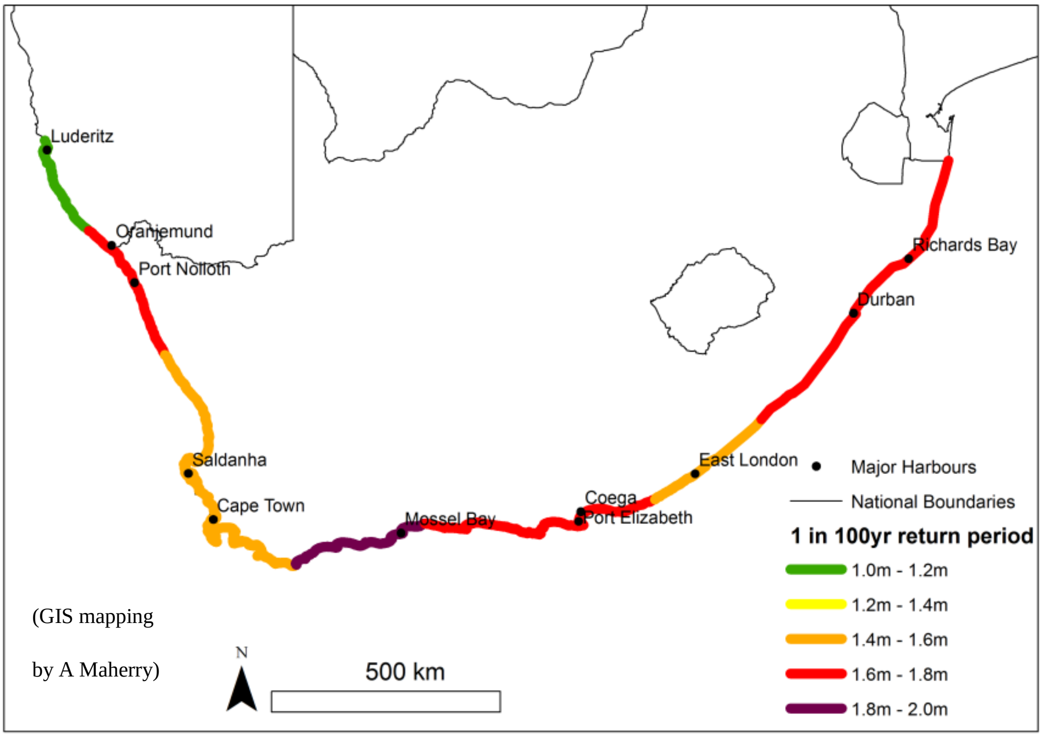
Figures 5.4 (a) to (c) respectively depict the 1-in-10-year, 1-in-50-year and 1-in-100-year predicted sea level extremes around the South African coast in terms of the residuals excluding tides (relative to CD), based on the extreme value distribution fitted to the data recorded in the harbours. (For the same reason as explained before for Table 5.2, the figures indicate the residual values above CD due to the offset between CD and MSL being included in the data.)


Luderitz
Oranjemund
Port Nolloth
Saldanha
Cape Town
Mossel Bay
Coega
Port Elizabeth
East London
Durban
Richards Bay
Major Harbours
National Boundaries
1 in 10yr return period
1.0m - 1.2m
1.2m - 1.4m
1.4m - 1.6m
1.6m - 1.8m
1.8m - 2.0m
(GIS mapping
by A Maherry)
N
500 km

(a)


Luderitz
Oranjemund
Port Nolloth
Saldanha
Cape Town
Mossel Bay
Coega
Port Elizabeth
East London
Durban
Richards Bay
Major Harbours
National Boundaries
1 in 50yr return period
1.0m - 1.2m
1.2m - 1.4m
1.4m - 1.6m
1.6m - 1.8m
1.8m - 2.0m
(GIS mapping
by A Maherry)
N
500 km

(b)



(c)

Figure 5.4: Residual sea levels above chart datum for the (a) 1-in-10-year, (b) 1-in-50-year and (c) 1-in-100-year event

Interpretation and discussion

As already mentioned, the variations and extremes found in the South African sea level data record, mainly result from astronomical tides, atmospheric pressure variations and wind effects. Keeping in mind that all of the recorders are located within semi-enclosed or sheltered water bodies (i.e. harbours), it is clear that virtually no gravity wave effects (i.e. 'normal' wind and swell waves with periods ranging from about 2 s to 24 s) are included in the data. In other words, wave setup and wave runup are not included in the recorded extreme water level data. Wind effects (i.e. wind setup) can penetrate into harbours and to some degree are included in the extreme sea level data. However, depending on (mainly) the specific location of each recorder, the coastal configuration, the wind velocity and direction, and the duration of the event, the *maximum* possible wind setup in the vicinity is usually not captured in the water level data. Wind setup is discussed further in Section 5.2.3. Thus, besides some degree of wind effects, the main components of extreme sea level included in the recorded water level data are the tides and barometric effects. Although the barometric effects are discussed further in Section 5.2.2, the best estimates of the extreme sea levels (based on long-term recordings) have been given in the foregoing parts of this section (Section 5.2.1), and these already include the barometric effects. (Although data inaccuracies and analyses problems are sometimes found in water level recordings, this judgement of "best estimates" is also based on the fact that *ratified* sea level data sets covering periods of about 45 years were used in the analyses, and that other theoretical means of estimating water level residuals are considered to yield less accurate results.)

5.2.2. Inverse barometric setup

Raised inshore seawater levels result from the effects of low local atmospheric pressure over the ocean. The pressure setup can be estimated by using the inverse barometer approximation, which translates to an increase of about 1 cm for every 1 hPa decrease in atmospheric pressure (Van Ballegooyen, 1996). This effect can thus be calculated by means of the following relationship (in lieu of applying a detailed numerical storm surge model):

$$\Delta S_p = (P_1 - P) * C$$

where ΔS_p = storm surge value (setup) due to inverse barometric effect (cm)

P_1 = 1 013 hPa (average sea level pressure)

P = forecast or observed (local) sea level pressure

C = 1 (pressure constant)

For the South African south-west coast, it is thus calculated that during typical winter storm events, severe additional rise above average water level due to localised low atmospheric pressure (also referred to as hydrostatic setup) is about 0.35 m (CSIR, 1987a). Considering the South African east coast and the typical passage of low-pressure cells, one finds that severe hydrostatic setup in this region is approximately 0.2 m to 0.3 m (i.e. approximately annual extremes). However, as discussed in Section 5.2.1, the main components of extreme sea level included in the water level recorder data are the tides, barometric effects and to some degree wind effects. Thus, the best estimates of the contribution to extreme sea levels due to barometric effects (and to some degree wind setup) have already been given before in Section 5.2.1.

Very strong cyclones (with very low central pressures) have been recorded along the Mozambican coastal region. Annual minimum pressures off the Mozambican coast (due to cyclones) are in the order of 100 hPa below the average sea level pressure, estimated from the Joint Typhoon Warning

Center data (JTWC, 2009). Thus, the annual maximum hydrostatic setup along the Mozambican coast is usually in the order of about 1 m. However, cyclone occurrence statistics in the Mozambique offshore region at present show an occurrence of about one third less in southern Mozambique relative to central Mozambique (Theron *et al*, 2012). In addition, very few cyclones approach farther south close to the South African coast (in the order of only one per decade in northern KZN), and even when they do, they have usually lost much of their strength by the time that they enter South African waters. Thus, additional provision for hydrostatic setup due to cyclones along the South African coast, over and above the hydrostatic setup component already included in Section 5.2.1, is extremely unlikely and deemed overly conservative.

5.2.3. Wind and wave setup

Extreme wind conditions can cause additional elevation of the local seawater level (Jury *et al*, 1986). This can be further exacerbated by the locally raised water levels caused by a low-pressure weather system. Wind setup at the coast due to onshore or alongshore winds is mainly a function of the slope of the seafloor and the wind speed. Wind setup of more than 0.15 m is not uncommon along the south-eastern coast of South Africa. This phenomenon is amplified if the wind blows into a semi-enclosed bay such as False Bay (e.g. Figure 5.5), which is a large bay of about 35 km by 35 km in size, situated near Cape Town. For an onshore wind of 25 m/s (which is likely to occur a few times every year in the south-western Cape), a wind setup of 0.5 m was predicted at the shore of False Bay (CSIR, 1987a). Along the South African east coast, maximum onshore wind speeds (with sufficiently long durations) could perhaps annually exceed 20 m/s. Thus, annual wind setup in the order of 0.2 to 0.3 m is considered likely for this region. According to Toms (2014), combined extreme wind and barometric setup along the South African coast is in the order of 0.3 m to 0.7 m. Raposeiro *et al* (2013) used a constant value of 0.52 m for combined extreme wind and barometric setup along part of the Portuguese coast (which experiences relatively similar conditions as part of the South African coast). Wind setup is frequently a lesser component of combined extreme inshore seawater levels, and along open coasts it can be relatively small (the amount is dependent on the shape of the coast) compared to extreme water levels due to other effects.

Figure 5.5: Effects of extreme onshore winds in Northern False Bay *(Photo R Klein)*

Relatively large waves often cause significant elevation of the local seawater level (Jury *et al*, 1986). (Wave setup is defined as the time averaged super-elevation of the water surface over normal water elevation near the shoreline due to onshore mass transport of the water by wave action alone.) Similar to wind setup, this phenomenon is also amplified if the waves propagate into a semi-enclosed bay such as False Bay. For the 1in-50-year wave height, the wave setup was calculated to be 1 m in False Bay (CSIR, 1987a). It is difficult to separate the wind setup from the wave setup and especially the usually more dominant wave runup. Various authors do not clearly distinguish between the wind setup and other wave-related water level increases, and some assume that the combined calculation of wave setup and wave runup includes the component of wind setup. If specific additional provision is made for wind setup, the combined total setup (wind + wave) tends to be somewhat overestimated. In addition, as discussed in Section 5.2.1, wind setup is also included to some degree in the recorded sea level data. For these reasons, the wind setup effect is assumed to be included in the calculation of total storm surge levels, which include water level extremes due to tides, residuals (hydrostatic and some wind setup), wave setup, plus potential SLR, as discussed further in this section and Section 5.2.5.

Various guidelines are provided in the literature to estimate the amount of wave setup at the coast. According to the Federal Emergency Management Agency (FEMA, 2000), the setup is 10–20% of the breaker wave height. The World Meteorological Organisation (WMO, 1988) states, “As a general rule of thumb, wave setup at the coast is about fifteen to twenty per cent of the incident root-mean square wave height." Guza and Thornton (1982) found the wave setup to be proportional to the significant wave height at 10 m depth (Hs_{10}) and estimated setup as 0.17 of Hs_{10}. Priestly (2013) has shown that wave setup (Sw) is equal to $0.19H_b$ (breaker wave height), if shallow water conditions and a breaking index of 0.78 is assumed (and assuming solitary wave theory). Using the same breaking criterion, Callaghan *et al* (2008) estimated maximum setup as $0.23H_b$. Karsten (2008) puts the setup at 20% of the offshore wave height (Hmo). Similarly, Dean and Dalrymple (2001) found the setup to be equal to 0.17Hso (significant offshore wave height). Taking all of these guidelines together it can be said that although they include significant differences and variations, a rough estimate of wave setup may be taken as between 10% to 23% of breaker wave height. Assuming extreme breaking wave heights of 5 to 9 m along exposed South African shorelines, this equates to roughly estimated wave setups in the order of 0.5 m to 2 m. Additional estimates based on alternative methods are provided in the following paragraphs to enable comparisons of results and to evaluate the consistency of such rough guidelines.

Stockdon *et al* (2006) suggested that wave setup (Sw) on *dissipative* beaches can be approximated (squared correlation $R^2 = 0.68$) by $0.016(Ho/Lo)^{1/2}$, where Ho and Lo are the deep water wave height and length. For wave periods (Tp) ranging from of 12 s to 18 s (typical South African storm wave conditions), Lo equates to 225 m and 506 m. Thus, for Tp ranging from of 12 s to 18 s, Sw = $0.24(Ho)^{1/2}$ to $0.36(Ho)^{1/2}$, which equates to 0.76 m to 1.14 m for a typical long-term *extreme* (1-in-10 year to 1-in-50 year) offshore (>100 m water depth) wave height of 10 m.

An approach presented by Goda (2000) can also be used to estimate the wave setup as a function of the wave height, period (Tp) and direction. According to Goda, the wave setup is 0.13 to 0.15 of the "equivalent unrefracted" offshore significant wave height (H'_0) for Tp up to 12 s, while for Tp above 12 s, the wave setup is 0.16 of H'_0. Along the South African coast, the wave periods associated with wave heights of 1-in-1 year or above, predominantly exceed 12 s (Patel and Moes, 2002). Thus, based on Goda's guidelines and the distribution of wave periods versus wave heights off South Africa, the wave setup factor is taken as 0.16. The "equivalent unrefracted" offshore significant wave height (H'_0) is related to the offshore wave height ($Hs_{offshore}$) by means of the refraction coefficient (Kr) as follows: $H'_0 = Kr * Hs_{offshore}$. Kr is mainly determined by the wave direction and period, as well as the relative orientation of the coastline. A matrix of simplified refraction coefficients for regions around the South African coast (delineated by overall orientation) is presented in Table 5.3 (Van Niekerk *et al* 2011). Note, that these refraction coefficients are derived for open coast locations and are not applicable inside bays or behind headlands.

Table 5.3: Simplified refraction coefficients (Kr) for regions around the South African coast *(Van Niekerk et al 2011).*

Wave direction		**Coastal region**				
		West	South-west	South	South-east	East
		Port Nolloth to Cape Town	Cape Town to Cape Agulhas	Cape Agulhas to Cape St Francis	Cape St Francis to East London	East London to Ponta do Ouro
NW	(315°)	0.74	-	-	-	-
WNW	(292.5°)	0.88	0.74	-	-	-
W	(270°)	0.95	0.78	0.50	-	-
WSW	(247.5°)	0.98	0.88	0.74	-	-
SW	(225°)	0.95	0.96	0.88	0.74	-
SSW	(202.5°)	0.83	0.96	0.95	0.83	0.74
S	(180°)	0.74	0.93	0.97	0.93	0.83
SSE	(157.5°)	-	0.83	0.95	0.97	0.93
SE	(135°)	-	0.74	0.88	0.96	0.97
ESE	(112.5°)	-	-	0.74	0.88	0.95
E	(90°)	-	-	0.50	0.78	0.88
ENE	(67.5°)	-	-	-	0.50	0.83
NE	(45°)	-	-	-	-	0.50

The simplified refraction coefficients (Kr) for regions around the South African coast thus almost all fall within the range of 0.5 to 0.98 (as noted they are not applicable inside bays or behind headlands). These simplified refraction coefficients account for wave refraction and shoaling, but do not include other effects such as bottom friction. The extreme wave conditions (all directions and locations) off South Africa (>100 m water depth) have been determined to range from 7.9 m to 11.1 m for 1-in-10-year events and from 9.3 m to 12.6 m for 1-in-50-year events, respectively (e.g. Rossouw and Theron, 2012). Thus, the "equivalent" unrefracted offshore significant wave height (H'_0 in >100 m water depth) ranges from approximately 4 m to 10.9 m for 1-in-10-year events and from 4.6 m to 12.3 m for 1-in-50-year events, respectively. By application of Goda's wave setup factor, the inshore wave setup is consequently estimated to range from approximately 1.2 m to 1.7 m for 1-in-10-year events and from 1.5 m to 2 m for 1-in-50-year events respectively (based on the deep-sea wave statistics used in this case).

In terms of regional differences, the highest wave setups occur in the Cape Columbine to Cape Agulhas area, while the lowest wave setups occur in the Orange River Mouth to Groen River Mouth and the Bashee River Mouth to Ponta do Ouro (South African West – and East Coast border) areas. Overall, the regional differences are not large, with the highest and lowest values differing by only about 0.5 m. The largest differences in wave setup indicated above (ca 0.5 m) are due to local wave exposure effects (i.e. the refraction coefficients mentioned above). Differences in SLR scenarios and wave runup are larger and thus more significant (as will be demonstrated in Sections 5.2.4 and 5.3). For these reasons, more accurate location-specific wave setups (which could theoretically be determined by means of detailed numerical wave modelling requiring detailed bathymetry data at each site), are not needed for this study on setback lines. Furthermore, it should be emphasised that wave setup is almost invariably included in wave runup determination, which is dealt with in detail in Section 5.3. This is in accordance with the basic understanding of these wave and water level phenomena and as applied by many authors, e.g. Stockdon *et al* 2006, Díaz-Sánchez *et al* 2013, Verhagen 2014, etc.

It is reiterated that the estimates provided in the foregoing paragraphs are based on relatively extreme events (in the order of 1-in-10 year to 1-in-50 year return period) and are applicable to open coast locations. For the purposes of conservative long-term coastal planning, including determination of setback lines, this is appropriate (on condition that the assumptions are applicable to the area being investigated). However, in view of the approximate estimation procedures (and some moderating factors such as bottom friction being neglected), the results may be deemed to be too conservative for other purposes; certainly the results are not suitable for detailed design. It should also be noted that within estuaries and harbours or sheltered areas (in the lee of headlands or capes, or behind islands), the wave setup phenomenon is mostly severely reduced.

5.2.4. Sea level rise

Recent observations from satellites, very carefully calibrated, are that global mean SLR over the last decade has been 3.3 +/- 0.4 mm/yr (Rahmstorf *et al*, 2007; Figure 5.6). The IPCC AR5 Report (IPCC AR5 SPM, 2013) concludes that anthropogenic warming and SLR would continue for centuries due to the timescales associated with climate processes and feedbacks, even if greenhouse gas concentrations were to be stabilised. Comparisons between about 30 years of South African tide gauge records and the longer-term records elsewhere show substantial agreement. A recent analysis of seawater levels recorded at Durban also found that the local rate of SLR falls within the range of global trends (Mather, 2008). Present South African SLR rates for the east coast are reported to be + 2.74 mm/yr-1 (Mather *et al*, 2009).

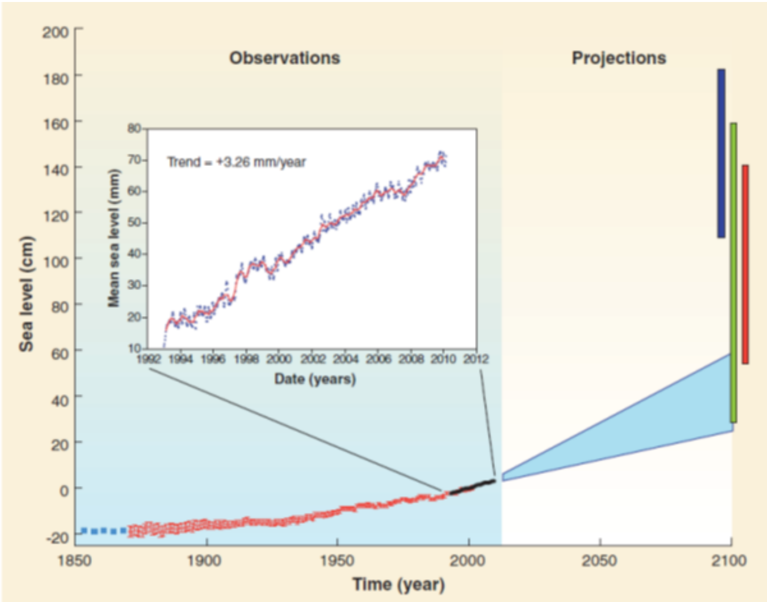


Figure 5.6: Measured and projected sea level rise *(Nicholls & Cazenave, 2010). (The blue, green and red bars are projections from different authors.)*

Sudden large rises in sea level (possibly several metres) due to catastrophic failure of large ice shelves (e.g. Church & White, 2006) are still considered unlikely this century, but events in Greenland (e.g. Carlson 2011; Gregory 2004; Overland, 2011) and Antarctica (e.g. Bentley, 1997; Thomas *et al*, 2004) may soon force a re-evaluation of that assessment. In the longer term, the large-scale melting of large ice masses is inevitable. Recent literature (post IPCC, 2007) gives a wide range of SLR scenarios, as indicated in Figure 5.7.

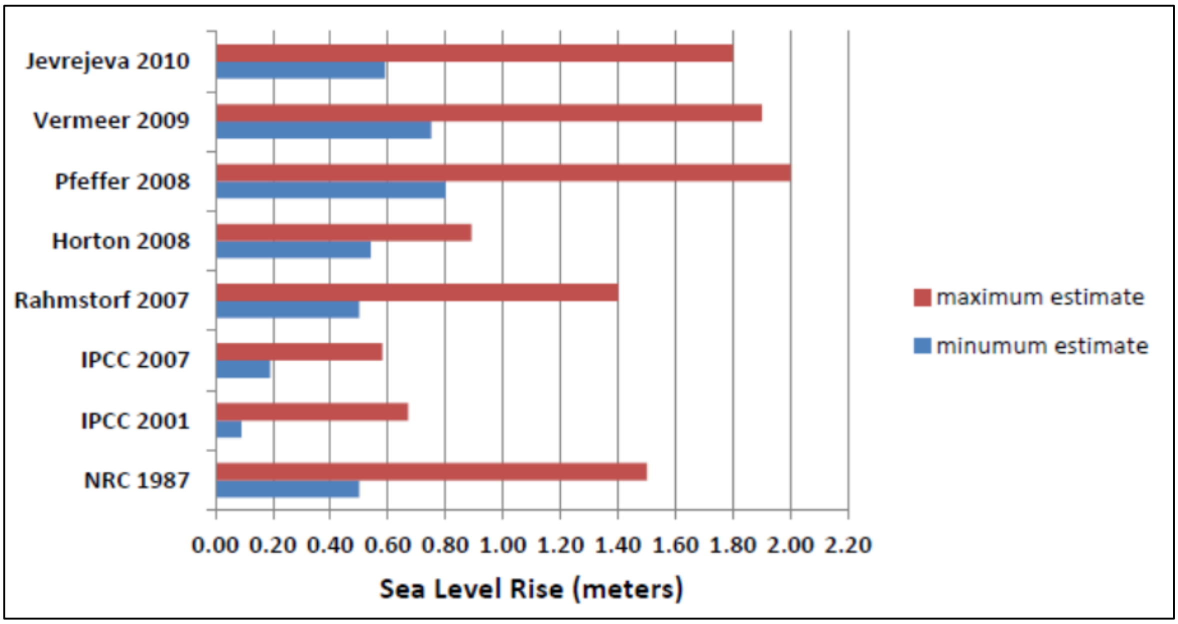


***Figure 5.7: Comparison of minimum and maximum estimates of global sea level rise by the year 2100** (USACE, 2011).* (Note: The post-2007 studies give an overall range of about 0.5 m to 2 m.)

Some projections and scenarios are even higher, but most "physics-" or "process-based" projections of SLR (e.g. Church *et al*, 2011; Milne *et al*, 2009; Nicholls & Cazenave, 2010; Pfeffer *et al*, 2008; SWIPA, 2011) for 2100 are in the 0.5 m to 2 m range, as is also concluded in various reviews (e.g. Fletcher, 2009; Theron *et al*, 2012; Theron, 2011; Parris *et al*, 2012 – Figure 5.8). One of the most recent authoritative documents is the IPCC report "Approved Summary for Policymakers" (IPCC AR5 SPM 2013). Projections of sea level rise are substantially higher than in the IPCC AR4 (2007) when comparing the same emission scenarios and time periods, primarily because of improved modelling of land-ice contributions. According to IPCC (2013), for the period 2081 to 2100, global mean sea level rise is likely (medium confidence) to be in the 5% to 95% range of 21 process-based models, which give from 0.26 m to 0.82 m (for the different scenarios: RCP2.6, RCP4.5, RCP6.0, and RCP8.5). For RCP8.5, which constitutes the "business as usual scenario" and therefore arguably the most relevant (or possibly the most likely) scenario, the rise by 2100 is 0.53 m to 0.97 m. However, the above process based projections do not include additional SLR contribution due to possible collapse of the Antarctic Ice Sheet, because the probability of this happening is currently undetermined. The potential additional contribution due to collapse of the Antarctic Ice Sheet cannot be precisely quantified but there is medium confidence that it would not exceed "several tenths of a meter" of sea level rise during the 21st century. Many semi-empirical model projections of global mean sea level rise (as included in the IPCC AR5) are higher than these process-based model projections (up to about twice as large, i.e. up to ca. 2 m SLR by 2100), but there is currently low confidence in their projections. So, the most appropriate IPCC AR5 range appears to be SLR of 0.3 m to 1.0 m by 2100 + “several tenths of a meter” (additional contribution for potential Antarctic Ice Sheet collapse with “medium confidence”); thus a total SLR of about 0.5 m to 1.2 m or more, but possibly even up to an *extreme* of 2 m. (It should be noted, that the “cut-off date” for literature considered for the AR5 report was during 2012 (IPCC AR5 SPM, 2013).) Later publications also seem to favour higher SLR projections with a total range in the order of 0.5 m to 2 m by 2100 (e.g. Rahmstorf *et al* 2012) depending on which scenario is considered. Jevrejeva *et al* (2014) state that their upper limit of 1.8 m for sea level rise by 2100, is based on both expert opinion and process studies, and advise that: “the upper limit of sea level rise is crucial for planning purposes in coastal areas, since infrastructure needs to survive the worst case situation”.

Based on all of the above literature and findings, it is concluded that an appropriate scenario for long-term coastal planning and setback lines is SLR by 2100 of ~ 0.85 m to 1 m (‘central estimate’), with a plausible worst-case scenario of 2 m and a low estimate of 0.5 m. The corresponding best estimate (mid-scenario) projections for 2030 and 2050 are about 0.15 m and 0.35 m, respectively. This is based on interpolation between the present values and the “target” values of 0.85 m and 1 m by 2100, as indicated by the curved blue lines added to the Parris *et al* scenarios by the author in Figure 5.8. It may be argued that lower scenarios are as plausible, but adopting significantly lower scenarios for the purposes of setback lines is not considered prudent, as there appears to be no justification for assuming that lower scenarios are more likely.

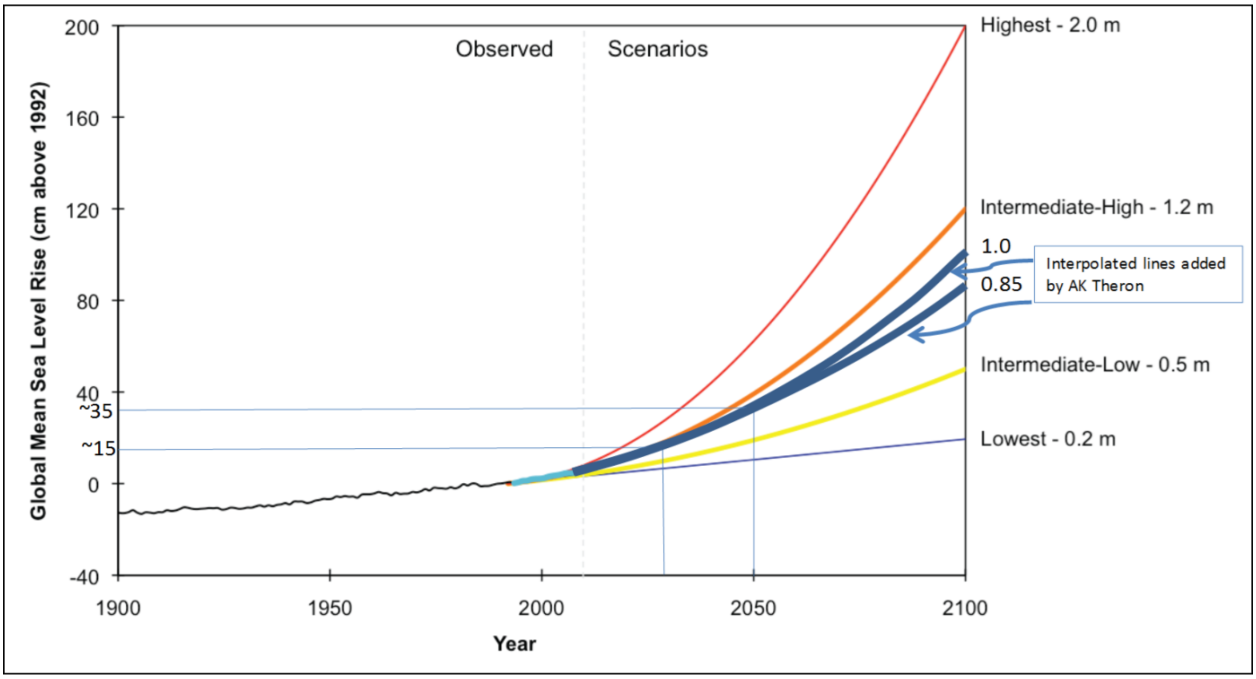


***Figure 5.8: Global mean sea level rise scenarios** (Parris et al, 2012, with additions by Theron).* (The Intermediate High Scenario is an average of the high end of ranges of global mean SLR reported by several studies using semi-empirical approaches. The Intermediate Low Scenario is the global mean SLR projection from the IPCC AR4 (2007) at 95% confidence interval.)

Planning horizons considered in determining setback lines are typically 50 years or 100 years. Thus it may be reasoned that taken from the present time, sea level rise scenarios are actually required for the year 2065 or 2115 for the respective 50 or 100 yr planning horizons. Based on the upper blue line in Figure 5.8, the approximate extrapolated sea level rise for the years 2065 and 2115 would then be 0.5 m, and 1.2 m respectively.

5.2.5. South African storm surge levels

In this section, estimates are made of extreme values for realistic combinations of the inshore seawater level components described in the foregoing Sections 5.2.1 to 5.2.4, as applicable to each South African coastal region. Based on these calculations and South African offshore wave conditions (e.g. Rossouw and Theron, 2012), estimates are made of the regional storm surge levels around the South African coast for the main offshore wave conditions. This provides a robust first-order coarse storm surge level assessment for the South African coastal regions. This feeds into coastal flooding determinations (Section 5.3.3), which also indicates the relative coastal flooding levels of the different South African coastal regions.

Sea levels along the South African coast are primarily affected by tide, waves (both sea and swell) and to a lesser degree long period waves (such as edge waves), but these processes have been found to be statistically independent (Wijnberg, 1993). According to Wijnberg (1993) the individual processes should be combined taking into account their particular characteristics of serial dependency and event structure (magnitude, duration and rate of occurrence). The joint probability of all of the considered events driving extreme inshore seawater levels occurring simultaneously, would be extremely low. To accurately determine the elevations for different return periods and account for joint probabilities should ideally be studied in an in-depth investigation outside of the present study. Conventional approaches to assess the joint effects of tides, surge, runup, etc. along shorelines are often based on multivariate statistical analyses or Monte Carlo simulations (such as provided by Hawkes *et al*, 2002), typically on a sub-set of the potential contributing drivers and processes. These methods can therefore provide statistical results linked to joint probabilities and recurrence intervals. However, these methods are computationally intensive and the assumption of independence of all the parameters is often questionable, as it is known that some of the phenomena driving extreme seawater levels are indeed often inter-related (i.e. not independent) (e.g. Alcock, 1993). Presently there seems to be no validated method to properly assess the joint probability of tides, surge, runup, etc. along shorelines (e.g. Lynett *et al* 2009). Wijnberg (1993) did develop a stochastic simulation model which reproduced a synthetic sealevel record displaying the same statistical characteristics as the data observed in three ports around the South African coast. As such, it does not simulate coastal phenomena such as wave runup or local wind setup. A new approach considering joint probabilities through "Archimedean Copulas" (Corbella *et al*, 2014) may lead to useful insights in this regard, but is currently still under development. The two methods presented by Mazas *et al* (2014), where POT methods are applied to the Revised Joint Probability Method for determining extreme sea levels due to tide and surge, seems good (although requiring a relatively high level of statistical expertise), but does not include wave runup, which is often a crucial component in South African study sites. (Wijnberg's (1993) probabalistic approach is somewhat similar.) Relatively long-term water level recordings, which include sufficient sea storm (or cyclone) events and resulting setups, are required to calculate statistically accurate extreme events and occurrences. This is supported by the recommendations of Wijnberg (1993), who called for further research on the possible interrelationships between the driving processes, and stated that more comprehensive field data would be required in order to investigate this problem. Unfortunately, such data for South Africa is still insufficient (as is the data on long period waves, such as edge waves). Therefore, following the precautionary approach, plausible scenario combinations are applied here, which is considered a first level approximation in that no accurate recurrence levels can be attributed to such straightforward combinations of events. Further research relating to inshore seawater levels in South Africa is required to address the questions of: which metocean events and physical coastal processes are related, how are they related, and to what degree do these joint occurrences result in or affect extreme coastal flooding levels.

In South Africa spring tides occur every two weeks, which means that the chances of storm waves coinciding with spring high tides are relatively high. Therefore, the input water level is set at spring high (MHWS) in the storm surge determination. A probable scenario would thus be to add the three setup effects, namely hydrostatic, wind and wave setup (which do have a serial dependency and sometimes occur in the same storm event although usually not totally simultaneously), to the mean high-water spring tide. In effect, this means combining Table 5.1 (spring high tides) and Table 5.2 (residuals) with the setup as determined according to Section 5.2.3. (This is basically the same approach recently followed by several other authors, e.g. Raposeiro *et al* (2013), who also added wind and barometric setup to mean high water spring levels, before then finally adding in wave runup which included wave setup.) If the three SLR forecasts of 0.15 m, 0.35 m and 1 m by 2030, 2050 and 2100 respectively (within "best/mid-SLR scenario predictions", Section 5.2.4) are then also added, probable maximum present (2014) and progressive future storm surge levels are predicted. (Similar results are obtained if HAT (in an 18.6-year cycle) is combined with only two of the three setup effects and SLR is added.) The South African storm surge levels thus calculated for each coastal area (i.e. combined mean high-water spring (MHWS) + wind, wave and atmospheric setup) for 1-in-10-year wave height and residuals (as an example) are indicated in Table 5.4. For example, to calculate the extreme wave setup for the Orange to Groen River Mouths, the 1-in-10 year offshore wave height of 8.3 m is multiplied by the refraction coefficient for this region (0.98 for WSW waves as per Table 5.3) and also multiplied by the calculated Goda factor of 1.6 (as per Section 5.2.3), which yields the result of 1.3 m as given in Table 5.4. The same procedure was followed to calculate the South African storm surge levels for each coastal area for 1-in-50-year wave height and residuals (i.e. combined MHWS tide + wind, wave and atmospheric setup).

***Table 5.4: Calculation of South African open coast storm surge elevations** (combined mean high-water spring (MHWS) + wind, wave and atmospheric setup for 1-in-10-year wave height and residuals)*

Coastal area	**Offshore wave height (1 in 10 year m)**	**Calculated extreme wave setup (m)**	**Tide (MHWS m to MSL)**	**Residual (baro., etc. 1 in 10 year m)**	**Total combined inshore seawater level (m above MSL)**			
				Year	**2013**	**2030**	**2050**	**2100**
				Sea Level Rise (m)	**0**	**0.15**	**0.35**	**1**
Orange River Mouth to Groen River Mouth								
	8.3	1.3	0.99	0.39	2.7	2.8	3.0	3.7
Groen River Mouth to Cape Columbine								
	10	1.6	0.89	0.72	3.2	3.3	3.5	4.2
Cape Columbine to Cape Agulhas								
	11.1	1.7	0.92	0.52	3.2	3.3	3.5	4.2
Cape Agulhas to Cape St Francis								
	10.7	1.7	1.17	0.86	3.7	3.8	4.0	4.7
Cape St Francis to Bashee Mouth								
	9.3	1.4	1.10	0.79	3.3	3.5	3.7	4.3
Bashee Mouth to Ponta do Uoro (South African border)								
	7.9	1.2	1.10	0.59	2.9	3.1	3.3	3.9

Similar to before, it should be noted that the estimates provided in Table 5.4 are based on relatively extreme events (1-in-10 year return period) and are applicable to open coast locations. The values are not applicable within estuaries and harbours or sheltered areas (in the lee of headlands or capes, or behind islands), where the wave setup phenomenon is mostly severely reduced.

To illustrate the severity of different return period events, the 1-in-10-year and 1-in-50-year return period wave conditions along each coastal region are considered in combination with the other sea level setup effects as described in the foregoing. The results, which are only applicable in the open coast areas, are summarised in Figures 5.9 to 5.14. The storm surge scenarios shown in Figures 5.9 to 5.14 are as follows:

- Figure 5.9: mean high-water spring (MHWS) + wind, wave and atmospheric setup along open coasts for 1-in-10-year wave height + 0-m SLR (no SLR assumed at present-day 2013).
- Figures 5.10 and 5.11: similar to Figure 5.9 but including progressive SLR scenarios of 0.35m and 1 m by 2050 and 2100 respectively (within "best/mid-SLR scenario predictions", Section 5.2.4).
- Figure 5.12: MHWS + wind, wave and atmospheric setup along open coasts for 1-in-50-year wave height + 0-m SLR (no SLR assumed at present-day 2013).
- Figures 5.13 and 5.14: similar to Figure 5.12 but including progressive SLR scenarios of 0.35m and 1 m by 2050 and 2100 respectively (within "best/mid-SLR scenario predictions", Section 5.2.4).

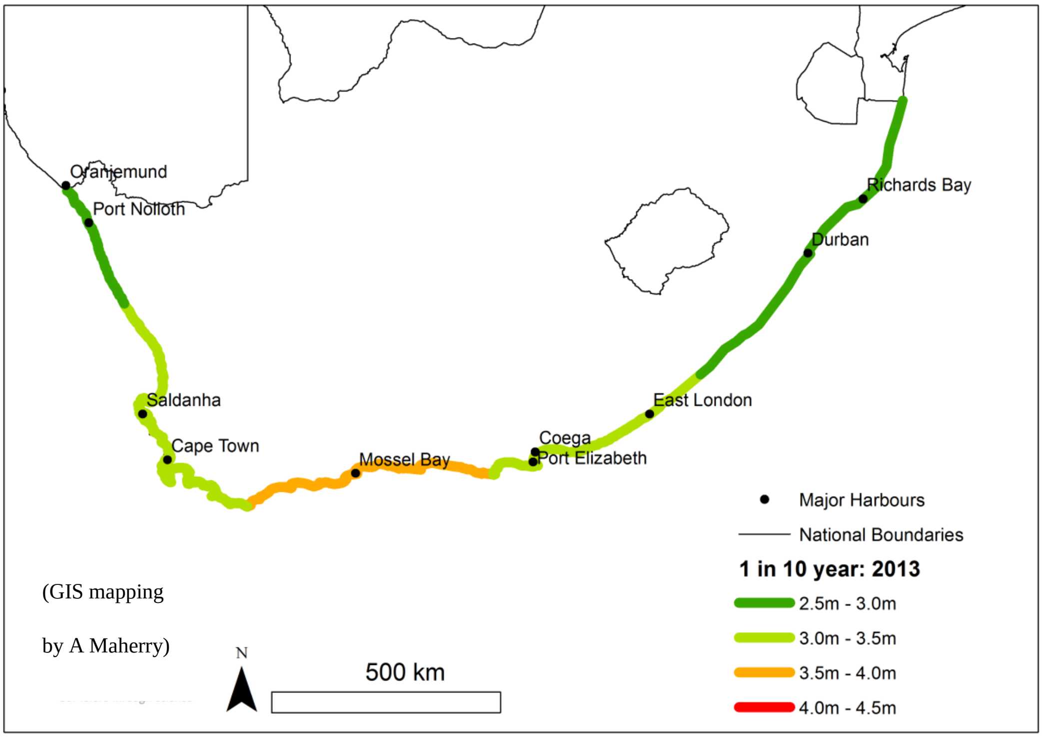


Figure 5.9: South African regional coastal storm surge elevations along open coasts for the 1-in-10-year wave return period and present day sea level (i.e. excluding wave runup)

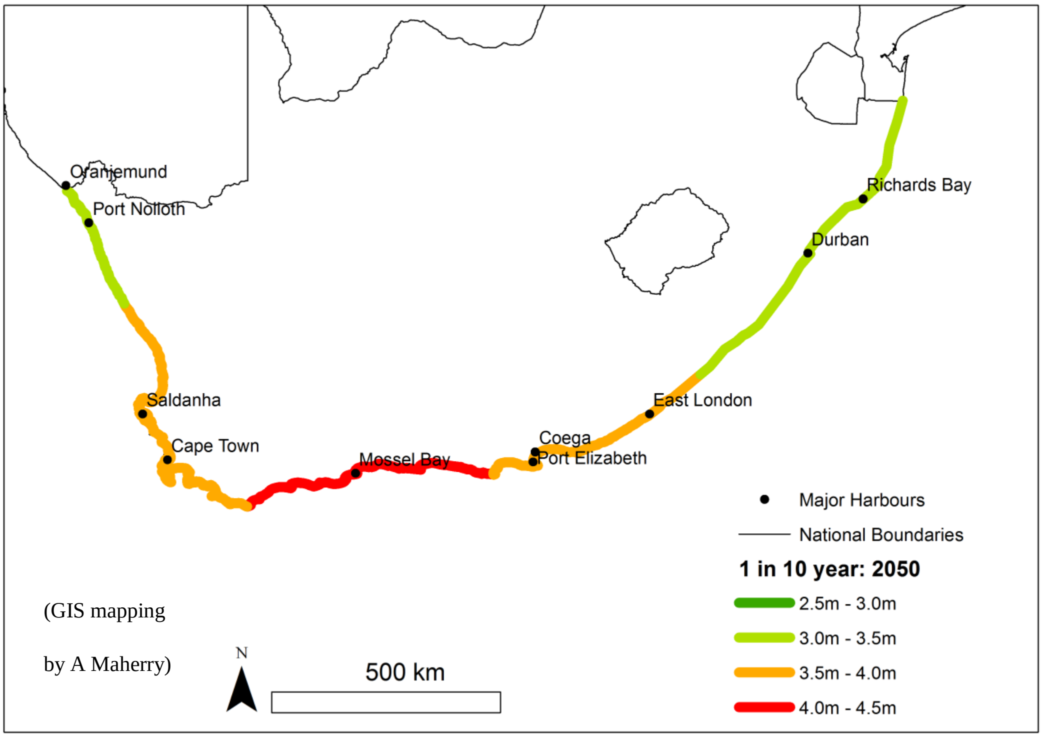


Figure 5.10: South African regional coastal storm surge elevations along open coasts for the 1-in-10-year wave return period and 0.35-m sea level rise scenario (i.e. excluding wave runup) at 2050

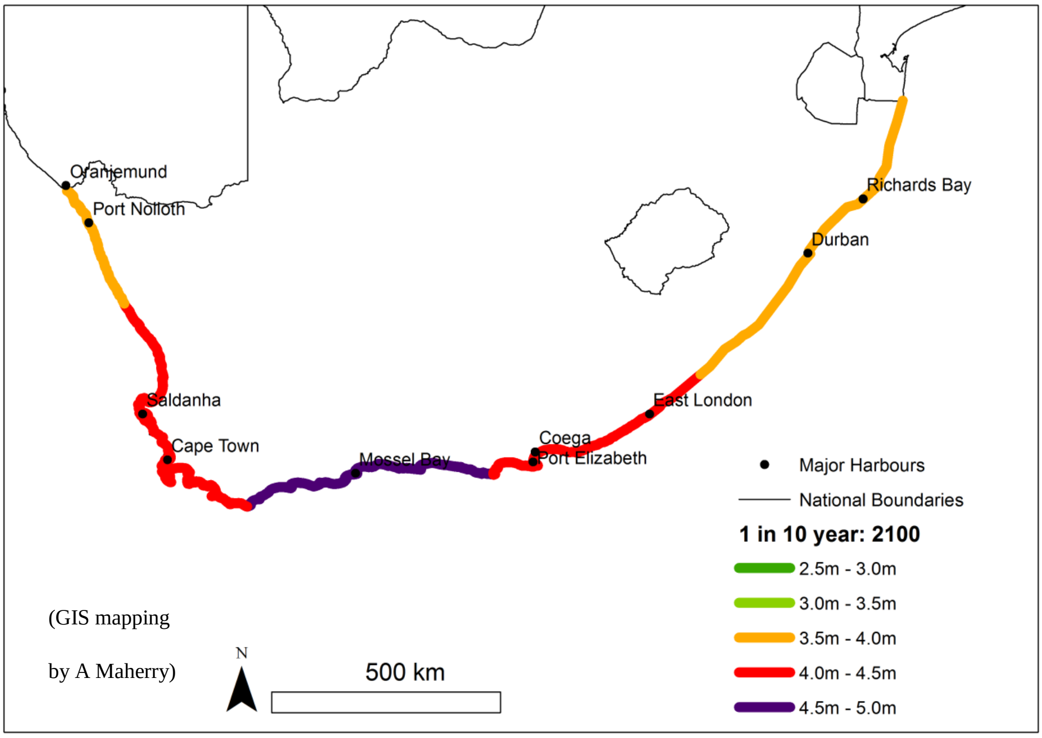


Figure 5.11: South African regional coastal storm surge elevations along open coasts for the 1-in-10-year wave return period and 1 m sea level rise scenario (i.e. excluding wave runup) at 2100

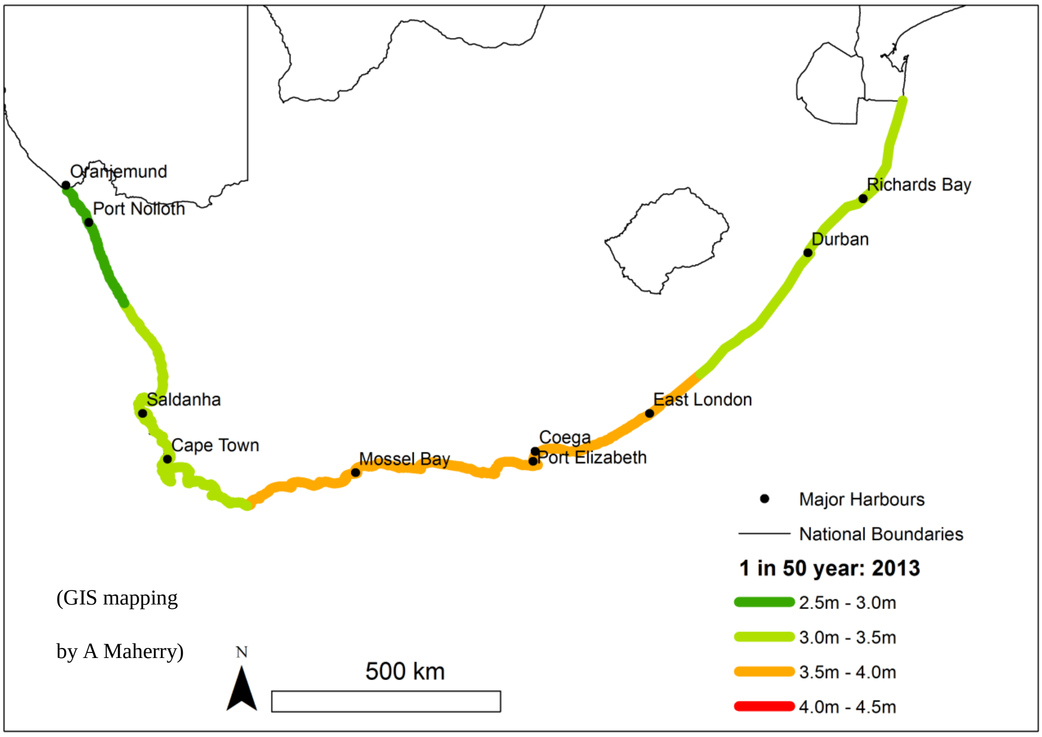


Figure 5.12: South African regional coastal storm surge elevations along open coasts for the 1-in-50-year wave return period and present day sea level rise (i.e. excluding wave runup).

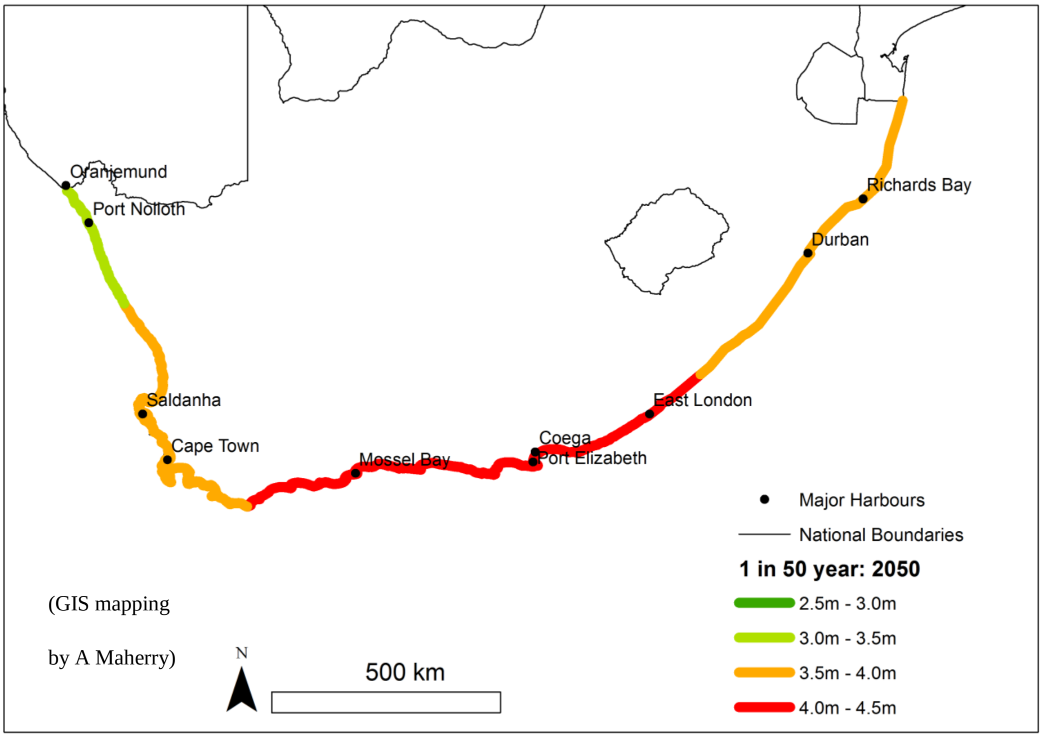


Figure 5.13: South African regional coastal storm surge elevations along open coasts for the 1-in-50-year wave return period and 0.35 m sea level rise scenario (i.e. excluding wave runup) at 2050

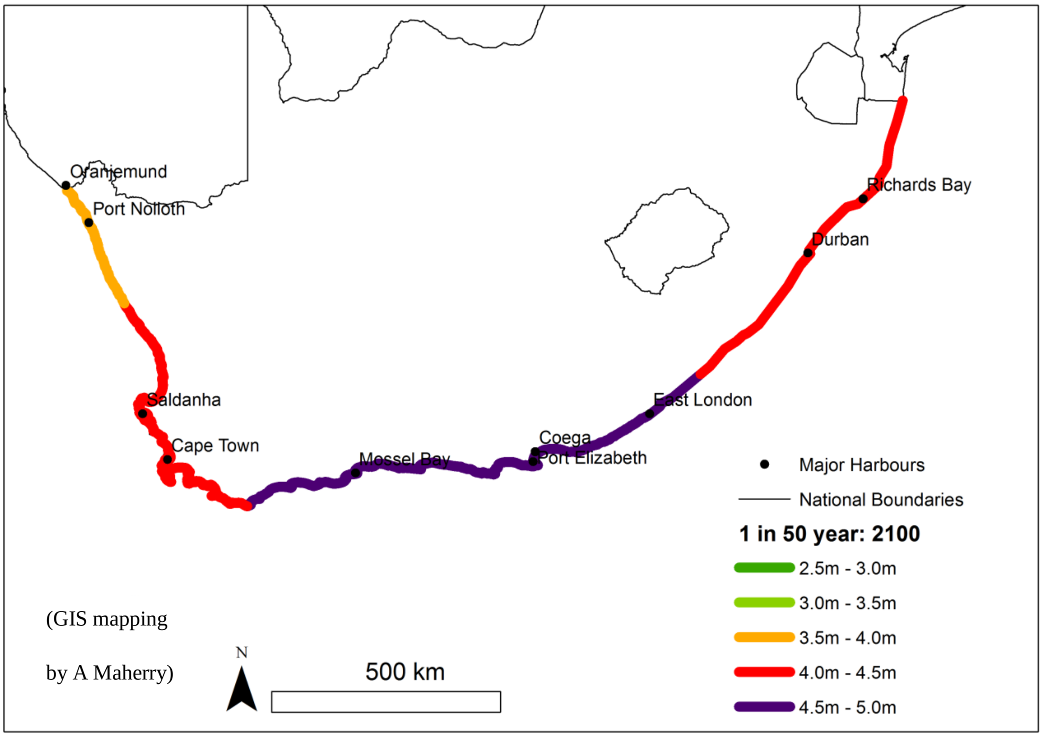


Figure 5.14: South African regional coastal storm surge elevations along open coasts for the 1-in-50-year wave return period and 1-m sea level rise scenario (i.e. excluding wave runup) at 2100

Note that the foregoing results are extreme open coastal "storm surge" levels (not applicable in bays) and do not include wave runup effects, as discussed in Section 5.3.

5.3. Wave runup

Wave runup is the rush of water up the beach slope beyond the still-water level (i.e. the swash zone) and usually results in the highest elevation of coastal flooding (and often impacts) along the South African coast. (As defined at the beginning of this chapter in Section 5.1, this refers to the short-term incursion of surface seawater over time scales ranging from seconds to minutes, and does not imply longer term inundation of areas below this elevation.) Wave runup prediction and the combined effects of waves, tides, water level setups and climate change effects are addressed in this section.

5.3.1. Evaluation of wave runup prediction methods

Evaluation

An important step in quantifying coastal flooding levels and in calculating setback lines (i.e. adequate development setback distances), is the determination of wave runup, in other words the maximum point that storm waves can reach (Figure 5.1). The wave runup is mainly a function of parameters such as wave height, direction and period, the surf zone width, the type of wave breaking, the roughness, slope and permeability of the near- and inshore profile (e.g. rocks or sand), the shape of the beach profile and the wave height distribution (Battjes, 1974). In detailed investigations of small study areas comprehensive wave and hydrodynamic modelling (ranging from two-dimensional to Boussinesq models) can potentially be employed to estimate wave runup elevations. Since complete hydrodynamic modelling of wave runup is impractical in most cases along the South African coast due to the lack of data on the input wind and wave conditions, accurate bathymetry and coastal topography, semi-empirical parameterizations are used to approximate the wave runup elevation. In the literature review of wave runup prediction methods (Section 2.5), 13 such methods were considered, namely: the model of Battjes (1971); that of Nielsen and Hanslow (1991); the formulation by Ahrens and Seelig (1996)); two formulations by Ruggiero *et al* (2001); Erikson *et al*'s version (2007) of a third model originally by Ruggiero *et al* (2001); the model of Guza and Thornton (1982); Mase's (1989) model, two models by Stockdon *et al* (2006); Priestley's (2013) version of Stockdon *et al* (2006), Diaz-Sanchez *et al* model (2013), and that of Mather *et al* (2010, 2011). The only locally (South African) derived model is a promising formulation proposed by Mather *et al* (2011). The model of Battjes (1971) was further developed by several of the later models evaluated below and was therefore not further evaluated. The formulation by Ahrens and Seelig (1996) was also not evaluated further, as it is considered impractical to apply, in that it requires input data that would almost invariably not be available (e.g. it requires the respective sediment sizes in both the surf and swash zones).

The remaining 11 models (i.e. excluding Battjes, 1971 and Ahrens and Seelig, 1996) were evaluated in more detail. The respective sets of formulations contained in these references (the afore-mentioned 11 runup models) were therefore used by the author in the compilation of computer routines, which were then tested against four sets of available field data. [Note, the respective formulations contained in the 11 models are available in the referenced literature; therefore only the 5 models ultimately selected for further testing are given in detail in the Section 5.3.2.] This field data was collected on four days in the Koeberg-Melkbos area (Cape West Coast) on two beaches having slopes (intertidal beach face) of 1-in-11 and 1-in-25 (Bartels, 1985), representing intermediate (but relatively steep) and dissipative conditions respectively. The highest runups in discrete 10 min periods were measured (by means of a survey level), tidal levels were recorded in an adjacent basin and waves were recorded in 22 m water depth off the study area (H_s ranged from 1.1 m to 3.6 m), while wind conditions (direction and strength) were also noted (Bartels, 1985). (For completeness and potential wider use, the full data set is tabulated in Appendix 2, Table A.) The wave runup elevations (based on the 2% runup elevation, R_2) predicted by each of the models are compared to the four sets of field data in Figures 5.15 to 5.20. R_2 is the level transgressed by 2% of the waves based on a Rayleigh distribution. Nielsen and Hanslow (1991) found that their data indicated that the Rayleigh distribution provides a reasonable description of the distribution of observed runup elevations. They also found that the measured maximum runup levels recorded in the field compared best with predicted 2% runup elevations (R_2) (as also reported by various other authors since then, for example Mather *et al*, 2011). Thus the norm has become to compare predicted R_2 (2% exceedance) values directly to measured maximum runup levels recorded in the field.

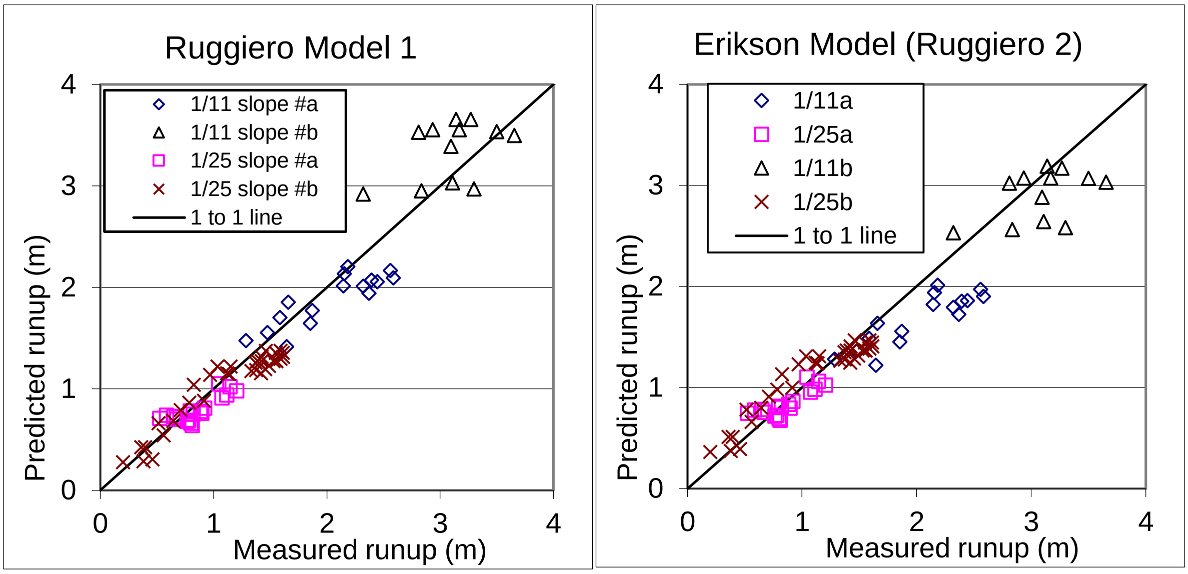


Figure 5.15: Predicted versus measured wave runup elevation – Ruggiero 1 (Ruggiero et al 2001) & Erikson (Erikson et al 2007) models.

Overall, the Ruggiero Model 1 performed reasonably well (R^2 = 0.93), and especially for the two dissipative field data sets (1/25 beach slope - Figure 5.15). The Erikson Model (which is the same as the second model proposed by Ruggiero) fared almost as well, but slightly less so for the two steeper sloped (1/11- Figure 5.15) field data sets.

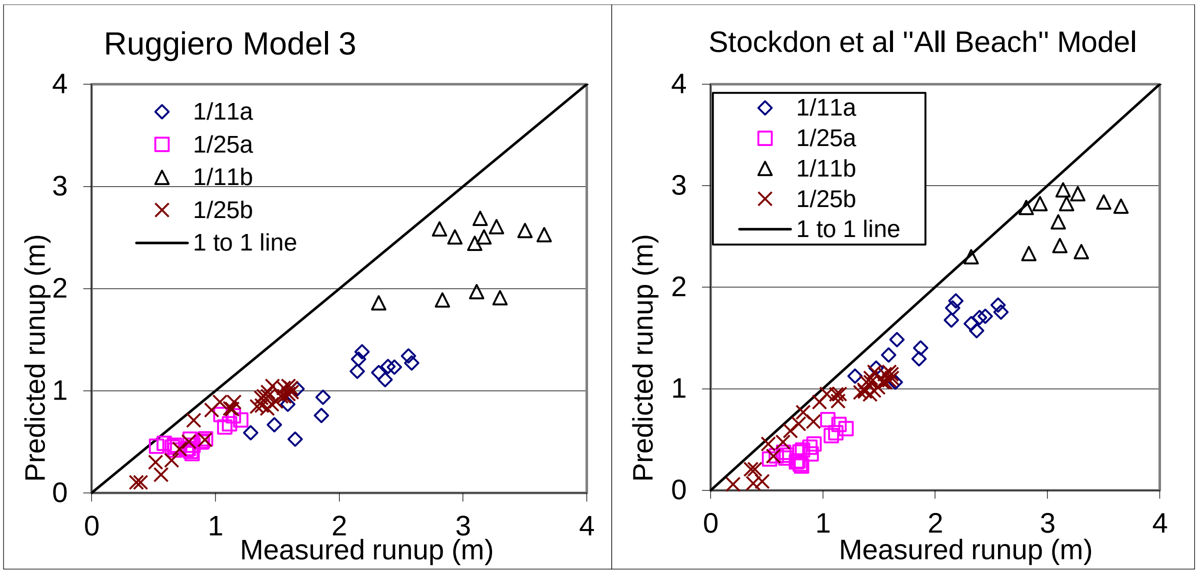


Figure 5.16: Predicted versus measured wave runup elevation – Ruggiero 3 (Ruggiero et al 2001) & Stockdon et al "All Beach" (Stockdon et al 2006) models.

The Ruggiero Model 3 (depicted in Figure 5.16) did not perform very well against the four data sets, especially for the two field data sets with steeper slopes. The Stockdon *et al* "All Beach" Model performed reasonably well (R^2 = 0.92) for all the field data sets (Figure 5.16).

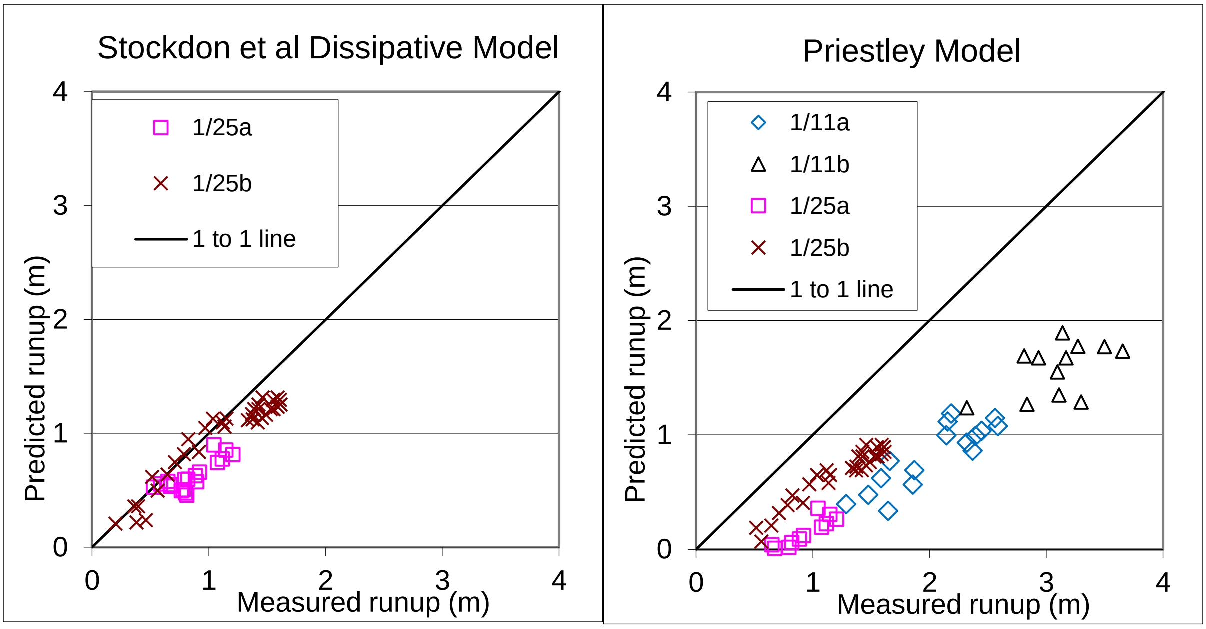


Figure 5.17: Predicted versus measured wave runup elevation – Stockdon et al Dissipative (Stockdon et al 2006) and Priestley (Priestley, 2013) Models

The Stockdon *et al* Dissipative Model gave relatively good results for the two field data sets collected under dissipative conditions (Figure 5.17). The model presented by Priestley (2013, as ascribed by Priestley to other researchers including Stockdon) did not perform well (Figure 5.17).

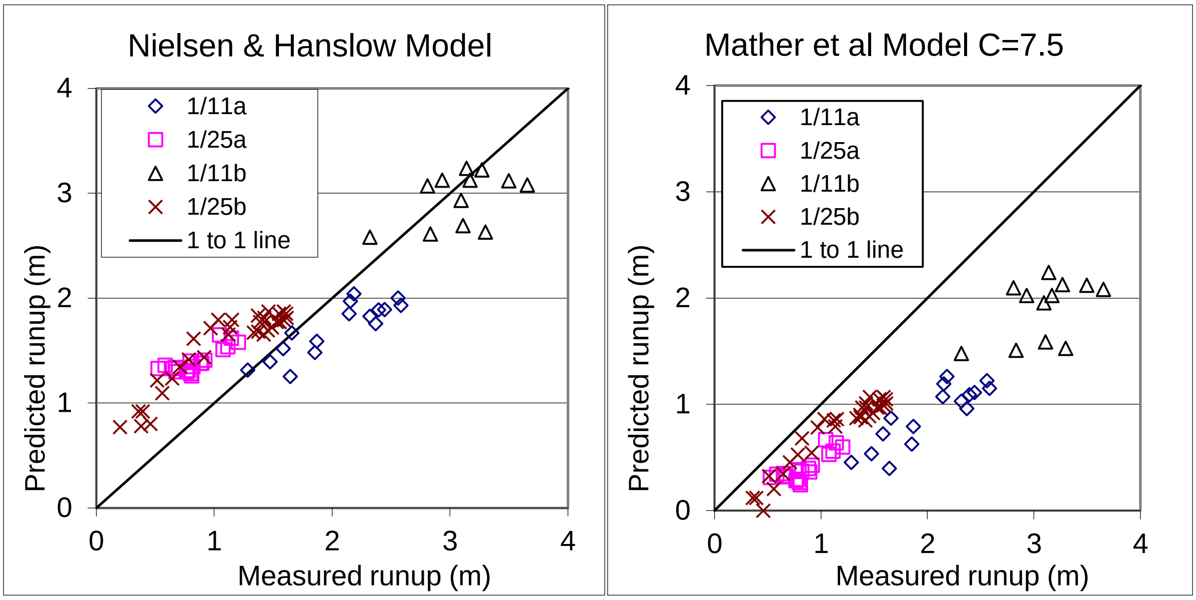


Figure 5.18: Predicted versus measured wave runup elevation – Nielsen & Hanslow (Nielsen and Hanslow, 1991) and Mather et al (Mather et al, 2010, 2011) Models

Overall the Nielsen and Hanslow (1991) Model performed reasonably well, especially for the two data sets from the steeper beaches (Figure 5.18). This model over predicted for the two milder sloped beaches (dissipative conditions). This is in line with the result shown by Mather *et al* (2011), where the Nielsen and Hanslow Model also generally over predicted for the data points from their 2007 KZN storm wave runup data set. The Mather *et al* Model under predicted in all instances (Figure 5.18), when applying the appropriate (open coast) value of 7.5 to coefficient C (see Mather *et al*, 2009 for a discussion on the appropriate values for C). This result could arguably be seen to be supported by the finding of Cariolet and Suanez (2013) who concluded that the use of the beach slope gives better results than using the foreshore slope which results in underestimates of runup values (based on studies of a macro tidal beach). However, although the "foreshore" slope extends deeper than the beach face slope, it does not extend as far as the slope to 15 m depth, which is that used in the Mather *et al* model.

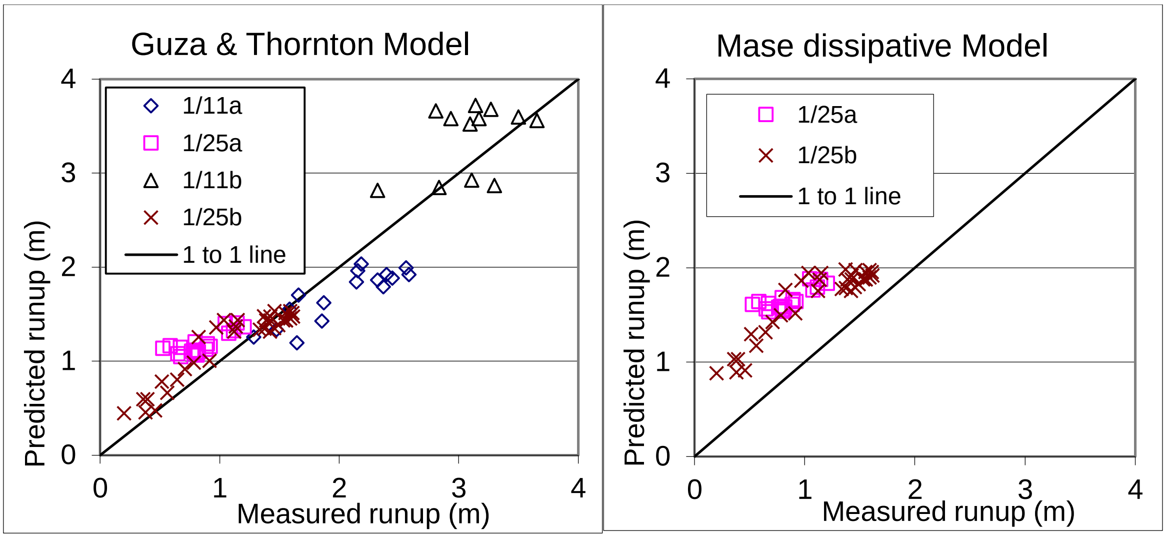


Figure 5.19: Predicted versus measured wave runup elevation – Guza & Thornton (Guza and Thornton, 1982) and Mase (Mase, 1989) Models

Application of the model of Guza and Thornton (1982) gave reasonable results, more so for the dissipative conditions (Figure 5.19), which are indeed the type of conditions for which this model is intended. However, it should be kept in mind that this model predicts R_s, which is the significant runup elevation (i.e. the average of the largest 1/3 of observed values) and different to virtually all of the other models which give the usual R_2 (2%) runup value. Power *et al* (2013) tested a number of wave runup models and concluded that Mase's (1989) model (as given in Power *et al*) gave the best results on gently sloped (dissipative) beaches (with tan $\alpha \leq 0.06$). However, in the present instance (Cape West Coast), Mase's model did not perform well for the two dissipative data sets (Figure 5.19).

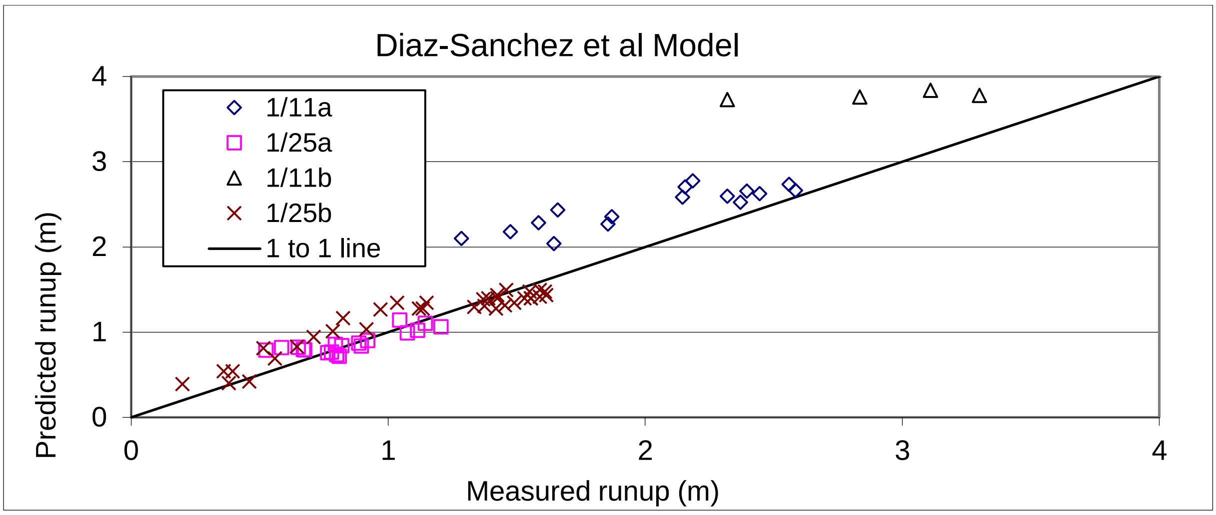


Figure 5.20: Predicted versus measured wave runup elevation – Diaz-Sanchez et al model (Diaz-Sanchez et al 2013)

The Diaz-Sanchez *et al* model (2013) performed reasonably well in all instances ($R^2 = 0.92$), but less so for the steep sloped beach conditions (Figure 5.20).

Discussion and selection of models for further testing

Taking all of the models assessed here into account, the best performer of all was the Ruggiero Model 1 (for both steep slopes and dissipative cases; overall $R^2 = 0.93$), and therefore clearly selected for further testing against field data representative of a much wider range of South African conditions.

In a very recent re-assessment of 9 wave runup formulae, Roux (2015) found the Stockdon *et al* (2006) "All Beach" Model to be one of the two best models. What makes Roux's findings particularly pertinent, is that he tested the models against a new (but albeit limited) South African field data set (from Long Beach near Cape Point) as well as against limited physical model data. Roux's findings are in line with the assessment by Mather *et al* (2011), who found the Stockdon *et al* (2006) Model to be the second best model for their wave runup data set. The Stockdon *et al* Model also has a stronger theoretical basis than most of the other less complex models, and did perform reasonably well for all the field data sets employed here (Figure 5.16). Therefore it was also selected for further assessment against a wider range of South African field data.

Mather *et al* (2011), demonstrated reasonably good results against their KZN runup data set, and found their model to be the best of the 5 models they tested against this data. Based on tests against his field data set from Long Beach (near Cape Point) as well as against limited physical model data, Roux (2015) also supported the Mather *et al* Model. Although it did not perform quite as well against the four South African West Coast data sets (Figure 5.18), the Mather *et al* (2011) model also has certain other advantages. The model can certainly be efficiently applied in large study areas and requires readily available input data, namely the offshore wave height and the distance to the 15 m depth contour (i.e. the near-shore slope). Many of the other models require the beach slope, which data is often not available. Furthermore, the Mather *et al* (2011) Model is the only locally (South African) derived model. Thus, this model was also selected for further assesment.

As mentioned, the Diaz-Sanchez *et al* model (2013) performed relatively well in all instances (Figure 5.20). It is interesting to note that both the Ruggiero Model 1 and the Diaz-Sanchez *et al* Model (2013), have the same form as the model originally proposed by Hunt (1959), the only real differences lying in the values of the coefficients or constants. In his recent re-assessment of wave runup formulae, Roux (2015) found the Diaz-Sanchez *et al* (2013) Model to be good against his physical model data. The Diaz-Sanchez *et al* (2013) Model is also the most recently developed of the 11 models tested here, and for these reasons was also selected for further assesment.

Roux (2015) found the Nielsen and Hanslow (1991) Model to be "best" against his South African field data, which is a strong recommendation. In the tests conducted here (Figure 5.18) the Nielsen and Hanslow (1991) Model faired reasonably well but not good (overestimated in the dissipative cases). In a review of a number of wave runup models, Power *et al* (2013), also concluded that Nielsen and Hanslow's model "was the only model that was accurate" on all of the contrasting Australian beaches investigated in their study, and that this model should be used where the beach slope (tan α) is greater than 0.06 (i.e. tan α > 0.06). The beach slope (tan α) refers to the beach face slope usually measured between about 0m MSL to +2 or +3 m to MSL (practically this is somewhat dependent on the available profile data). The focus here is on wave runup during extreme events (storms), which means that the beach profile between about 0 m MSL to +2 or +3 m to MSL is most probably being eroded during the event. In this case, the beach slope (tan α) would become steeper during the event. Thus, it may be argued that although a particular beach might typically (or on average) exhibit a mild beach slope (say tan α < 0.06), this slope is likely to become steeper during an extreme event (say tan α during the storm grows to > 0.1), which would result in a higher runup elevation. Although this might not necessarily always be true, the implication would be that the slope of beaches that typically (or on average) exhibit a mild beach slope could be assumed to be steeper during storms (and steeper than the slopes typically recorded during average conditions). This assumption would be subject to many conditions such as the particular site characteristics (e.g. wave exposure, sediment grain size, etc.), the particular conditions during such storm (e.g. storm duration, tidal level, wave conditions, etc.) and other factors (e.g. abundant sediment supply or deficit). Nevertheless, in situations where this assumption (of steeper slope during storms) is true, it would mean that the Nielsen and Hanslow model (1991) might still be applied, despite this model perhaps being generally less applicable for beach slopes less than 0.06. Based on these considerations, especially the good reviews found in some of the literature (e.g. Roux, 2015; Power *et al*, 2013), the Nielsen and Hanslow (1991) Model was selected as the fifth and final model for further assesment.

5.3.2. Final testing and adaptation of wave runup prediction methods

Final testing and adaptation of selected wave runup prediction models

Formulation of the wave runup prediction models

As concluded in the previous section, the wave runup models that were selected for further testing and validation against field data representative of a much wider range of South African conditions, were the Ruggiero *et al* (2001) Model 1, the Nielsen and Hanslow (1991) model, the Diaz-Sanchez *et al* model (2013), the Stockdon *et al* (2006) Model and the Mather *et al* (2011) Model. The formulae contained within these five models are described in the following paragraphs.

The Stockdon *et al* (2006) Model:

Based on field data from the Netherlands and the USA, Stockdon *et al* (2006) derived their model for the 2% exceedance wave runup (R_2) as follows:

$$R_2 = 1.1\left(0.35\beta_f(H_0L_0)^{1/2} + \frac{[H_0L_0(0.563\beta_f^2 + 0.004)]^{1/2}}{2}\right)$$

Where: β_f is the beach face slope (measured in the foreshore), H_0 is the deep water significant wave height, and L_0 is the deepwater wave length calculated from the peak wave period (Tp):

$$L_0 = \frac{gTp^2}{2\pi} \quad \text{(deepwater wave length)}$$

The Nielsen and Hanslow (1991) model:

The Nielsen and Hanslow (1991) model requires the deep water root mean squared wave height (H_{0rms}) and peak wave period (T_p), beach face slope (tan α, measured in the foreshore), gravitational acceleration constant (g = 9.81 m.s-1) and water level (WL) as input. Two different formulae were derived by Nielsen and Hanslow for the wave runup height (R_2 above MSL), depending on whether tan (α) is greater or less than 0.1.

For tan (α) greater than 0.1:

$$R_2 = WL + 1.98 \cdot (0.6 \cdot \tan(\alpha) \cdot \sqrt{\beta})$$

For tan(α) less than or equal to 0.1:

$$R_2 = WL + 1.98 \cdot (0.05 \cdot \sqrt{\beta})$$

Where:

$$\beta = \frac{H_{0rms}}{\sqrt{2}} \cdot L_0$$

and: $L_0 = \dfrac{gTp^2}{2\pi}$ (deepwater wave length)

The Ruggiero *et al* (2001) Model 1:

The Ruggiero *et al* (2001) Model 1 is a function of the deep water significant wave height and the Irribaren number:

$$R_2 = H_0 . (0.75 . Ir + 0.22)$$

Where: R_2 is the 2% runup elevation in m, and

$$Ir = \tan(\alpha) / (H_0 / L_0)^{0.5}$$

Where: Ir is the Irribaren number (as defined in many references, e.g. Diaz-Sanchez *et al* 2013), tan(α) is the beach face slope (measured in the foreshore), H_0 is the deep water significant wave height, and L_0 is the deepwater wave length calculated from the peak wave period (T_p):

$L_0 = \dfrac{gTp^2}{2\pi}$ (deepwater wave length)

The Mather *et al* (2011) model:

The Mather *et al* (2011) model uses the distance offshore (x_h) to the water depth (h) to estimate the near-shore profile slope as $S = h/x_h$, where the depth of closure is the suggested choice for the water depth h (nominally taken to be about 15 m). Extreme runup R_x is then expressed in terms of S as:

$$R_x / H_0 = C.S^{2/3}$$

R_x is the runup value, x_{15} is the chart distance from the shoreline (nominally taken as the MSL contour line) to the 15 m isobath, and H_0 is the deep water significant wave height. In the equation $R_x / H_0 = C.S^{2/3}$, C is a dimensionless coefficient (ranging from 3 to 10) that is used to predict wave runup based on 3 different coastline types (open coast (C=7.5), and large (C=5), or small (C=4) embayments; Mather *et al*, 2011). Thus, the model can be written as:

$$R_2 = WL + C.\,H_0.\,(15/x_h)^{2/3}$$

Where: R_2 is the 2% runup elevation in m above MSL, and WL is the input "still" water level above MSL (due to, for example, tides, barometric and SLR effects).

The Diaz-Sanchez *et al* model (2013):

The Diaz-Sanchez *et al* model (2013) is similar to the Ruggiero *et al* (2001) Model 1, and is also a function of the wave height and the Irribaren number:

$$R_2 = 1.4\,.\,H_0\,.\,Ir$$

Where: R_2 is the 2% runup elevation in m, and

$$Ir = \tan(\alpha)\ /\ (H_0 / Lo)^{0.5}$$

Where: Ir is the Irribaren number, $\tan(\alpha)$ is the beach face slope (measured in the foreshore), H_0 is the deep water significant wave height, and L_0 is the deepwater wave length calculated from the peak wave period (Tp):

$$L_0 = \frac{gTp^2}{2\pi} \text{ (deepwater wave length)}$$

Testing of the wave runup prediction models – Table Bay data:

Runup data along the Table Bay shoreline was collected by the CSIR following a major storm in 2008. The maximum significant wave height recorded during this storm (in 70 m water depth) was about 10.3 m which was calculated to have a return period of about 10 years. Still water levels were recorded at the Port of Cape Town, which is also located within Table Bay. (For completeness and potential wider use, the full data set is tabulated in Appendix 2, Table B.) The abovementioned five models were tested against this data set, as indicated in Figures 5.21 to 5.23. The accuracy of the model predictions were objectively assessed via the Root Mean Square Error of Prediction (RMSEP) which is defined as:

$$RMSEP = \sqrt{\frac{\sum_{i=1}^{n} (\hat{y}_i - y_i)^2}{n}} \ ,$$

where $\hat{y}_i$ is the predicted values from the model, y_i is the measured data values and n is the number of measured data points. (The RMSEP is given in meters and is thus also an easy to interpret manner of quantifying the model prediction error.)

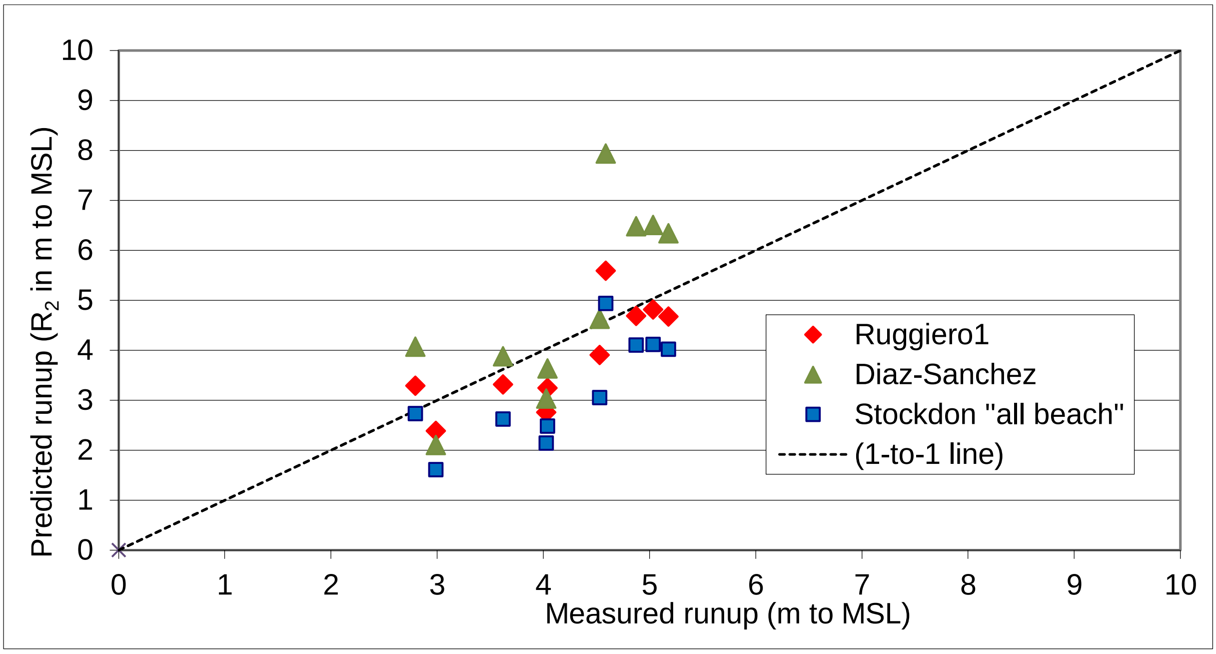


Figure 5.21: Predicted versus measured wave runup elevation – Ruggiero et al (2001) Model 1, Diaz-Sanchez et al (2013) Model, and Stockdon et al (2006) Model.

The Ruggiero *et al* (2001) Model performed well against this data set (Figure 5.21) with a Root Mean Square Error of Prediction (RMSEP) of only 0.68 m. The Diaz-Sanchez *et al* (2013) Model performed reasonably well (Figure 5.21), but both over- and under predicted significantly in respective instances (RMSEP = 1.45 m). The Stockdon *et al* (2006) Model also performed reasonably well (Figure 5.21), but under predicted significantly (RMSEP = 1.18 m).

The Nielsen and Hanslow (1991) model over predicted significantly (RMSEP = 2.78 m) with the offshore wave height (H_{0S} = 8.3 m) as input (Figure 5.22).

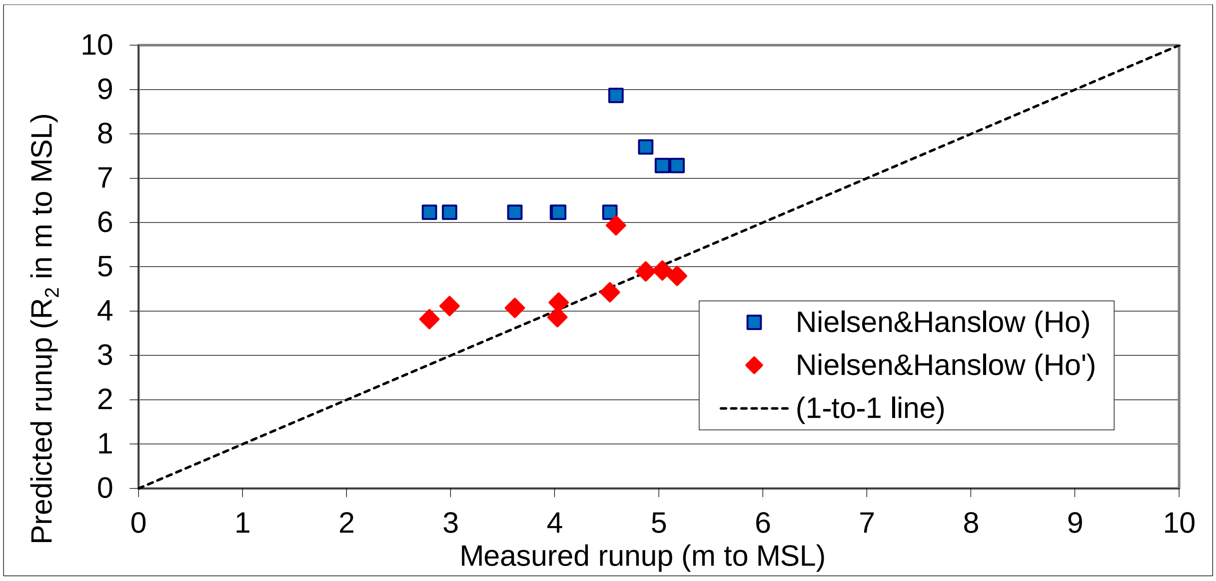


Figure 5.22: Predicted versus measured wave runup elevation – Nielsen and Hanslow (1991) Model

It was verified that the correct values were indeed used for these wave height and period, beach slope, and water level input parameters at each runup data location within Table Bay. Keeping in mind that in this instance the locations of the runup recordings are along the relatively sheltered shoreline within Table Bay, the offshore wave height is changed (reduced) significantly as the waves propagate into the bay (due mainly to refraction and shoaling). Wave refraction coefficients for Table Bay were available from a previous modelling study (Luger *et al* 2004). By applying these coefficients to the offshore wave conditions of the 2008 storm, the inshore wave heights were calculated at each of the wave runup data locations. These reduced inshore wave heights were then converted to “equivalent unrefracted” offshore wave heights (H_0’, ranging from about 3 m to 4.5 m, from south to north along the Bay) by applying the relevant (inverted) shoaling coefficient in each case. This is similar to the conclusion made by Stockdon *et al* (2006) that the best runup predictions were obtained with significant wave heights measured at 8 m to 18 m depth, and then “reverse shoaled” to give the equivalent deep-water wave heights. It is also interesting to note here that Guza and Thornton (1982) found wave setup to be proportional to the significant wave height determined at 10 m water depth, therefore in the same depth range. Thus, in the Table Bay case, the “equivalent unrefracted” offshore wave heights (H_0’) were then again applied in the Nielsen and Hanslow model, giving the second set of results also displayed in Figure 5.22 (as indicated by the diamond markers/symbols). Using these H_0’ wave heights as input, therefore resulted in good agreement (RMSEP = 0.67 m) with the recorded runup values being achieved (for both the steep and mild sloped cases in this data set).

Application of the Mather *et al* model (2011) to the 2008 Table Bay data set, yielded the results as indicated in Figure 5.23.

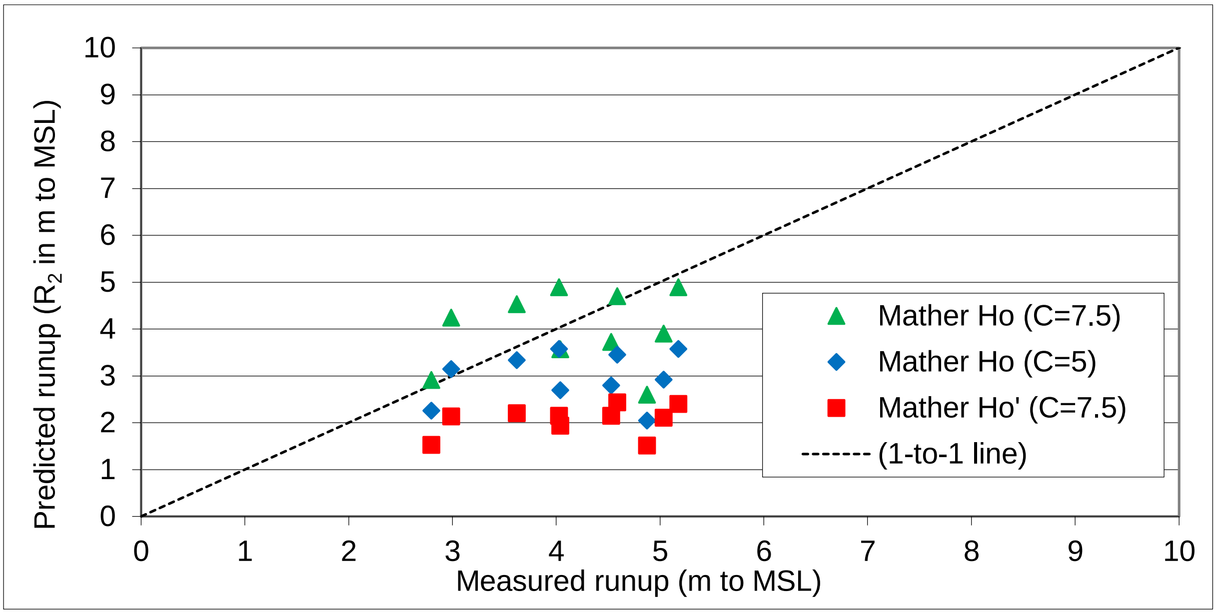


Figure 5.23: Predicted versus measured wave runup elevation – Mather et al model (2011)

Direct application of the Mather *et al* model with the unrefracted open coast deep-water wave height and the value of coefficient C set at 7.5, gave rise to reasonably good predictions (RMSEP = 1.16 m) for almost all of the locations (green triangles in Figure 5.23). Applying the same input wave height and setting the value of coefficient C at 5, resulted in significant under predictions in almost all locations (blue diamonds in Figure 5.23), even though this would appear to be the appropriate value of C in this case according to Mather *et al* (2011). Applying the "equivalent unrefracted" offshore wave heights (H_0') as derived before for the Nielsen and Hanslow model test, and with C set at 7.5, resulted in even larger under predictions in all of the locations (red squares in Figure 5.23). Thus, the Mather *et al* model indeed performed best with the direct open coast deep-water wave height as input (which is in accordance with the original model derivation), but with coefficient C set at 7.5.

Final testing of the wave runup prediction models – KZN data:

Runup data along the KZN shoreline (in the vicinity of Durban) was collected by the eThekwini Municipality following a major storm in 2007. (This is the same storm already discussed in Section 2.5). Still water levels were recorded at the Port of Durban. The full data set is tabulated in Mather *et* *al* (2011). For further testing and validation against field data representative of a much wider range of South African conditions, the selected five models (i.e. Ruggiero *et al* (2001) Model 1, Nielsen and Hanslow (1991) model, Diaz-Sanchez *et al* model (2013), Stockdon *et al* (2006) Model, Mather *et al* (2011) Model) were also tested against this data set. The results are graphically illustrated in Figures 5.24 to 5.28. The Root Mean Square Error of Prediction (RMSEP) was also calculated again to objectively assess the accuracy of the model predictions.

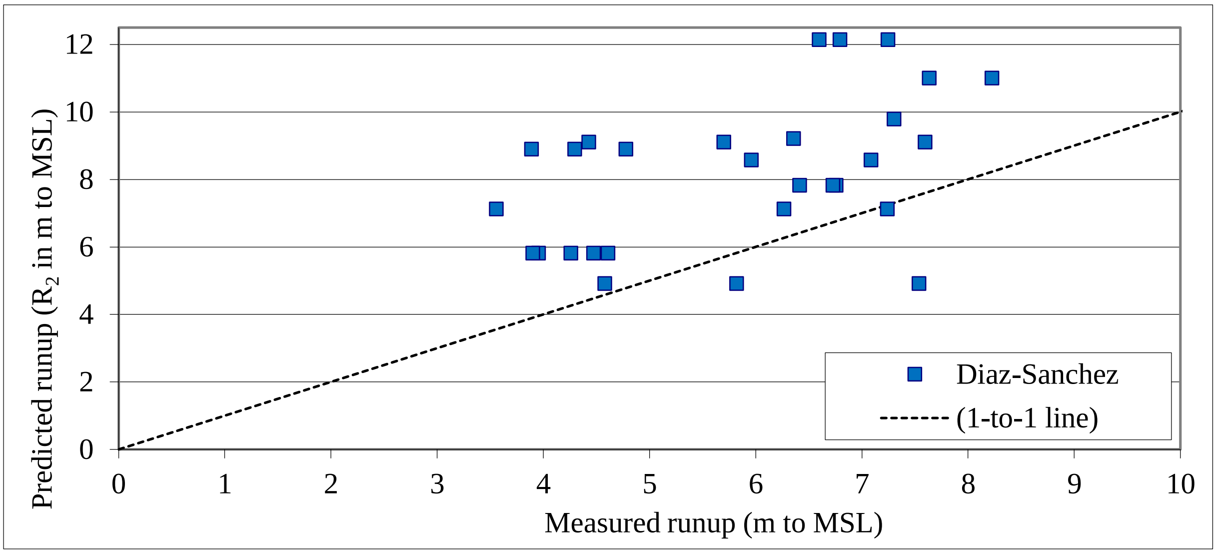


Figure 5.24: Predicted versus measured wave runup elevation (KZN) – Diaz-Sanchez et al (2013) Model.

The Diaz-Sanchez *et al* (2013) Model did not perform well against the KZN data (Figure 5.24), and mostly over predicted significantly (RMSEP = 3.01 m).

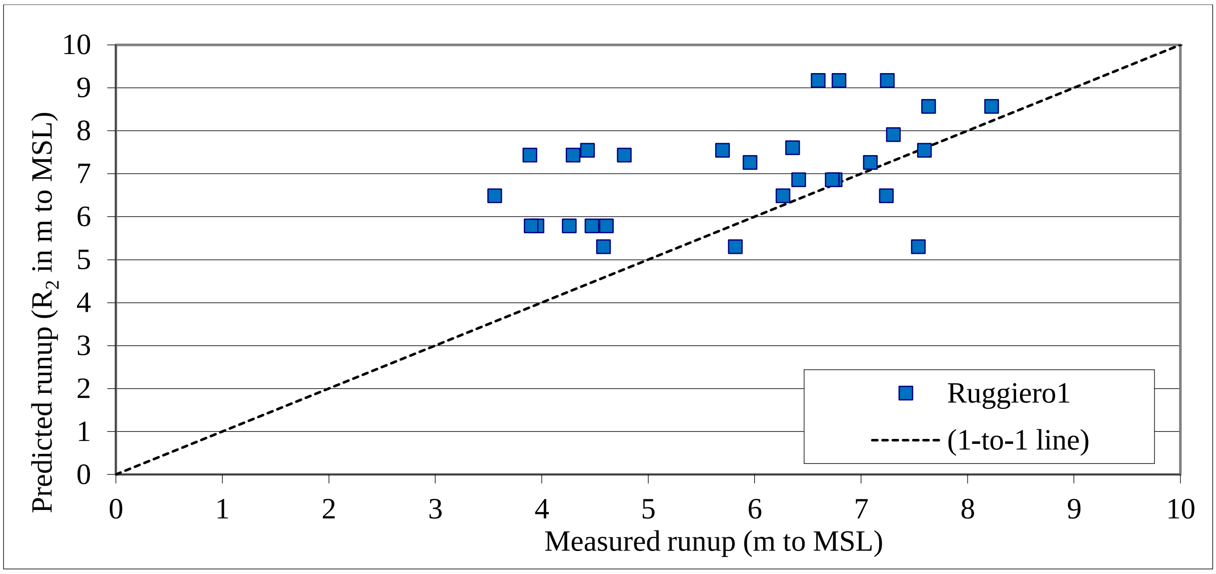


Figure 5.25: Predicted versus measured wave runup elevation (KZN) – Ruggiero et al (2001) Model 1.

The Ruggiero *et al* (2001) Model did not perform very well against this data set (Figure 5.25) with a RMSEP of 1.77 m.

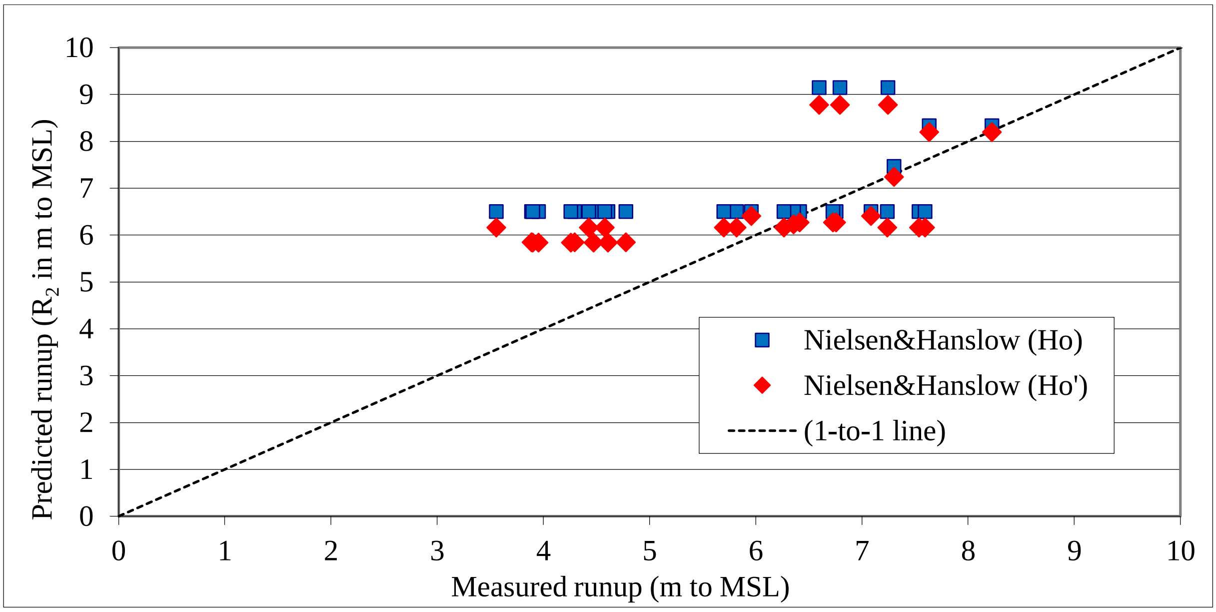


Figure 5.26: Predicted versus measured wave runup elevation (KZN) – Nielsen and Hanslow (1991) Model.

The Nielsen and Hanslow (1991) model over predicted significantly (RMSEP = 1.64 m) with the offshore wave height as input (Figure 5.26). Wave refraction coefficients for the Durban area were available from a previous modelling study (Theron *et al* 2014). By applying these coefficients to the offshore wave conditions of the 2007 storm, the inshore wave heights (at 15 m depth) were calculated at each of the wave runup data locations. These reduced inshore wave heights were then converted to "equivalent unrefracted" offshore wave heights by applying the relevant (inverted) shoaling coefficient in each case. This is similar to the procedure conducted before for the Table Bay data set. Thus, for the KZN case, the "equivalent unrefracted" offshore wave heights (H_0') were then again applied in the Nielsen and Hanslow model, giving the second set of results also displayed in Figure 5.26 (as indicated by the diamond markers/symbols). Using these H_0' wave heights as input, therefore resulted in significantly better agreement with the recorded runup values being achieved (RMSEP = 1.32 m).

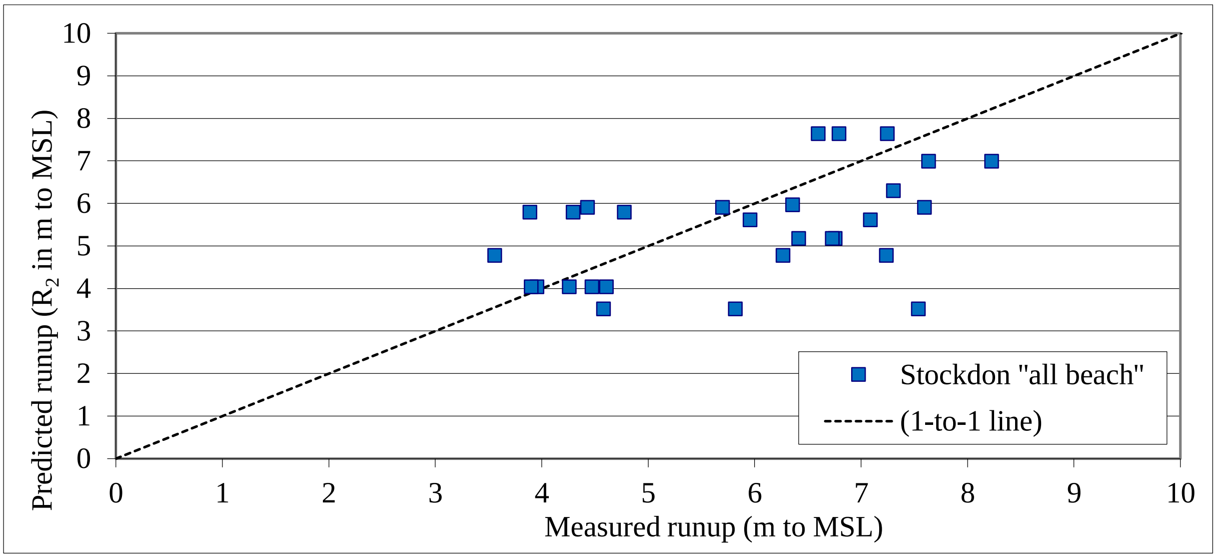


Figure 5.27: Predicted versus measured wave runup elevation (KZN) – Stockdon et al (2006) Model.

The Stockdon *et al* (2006) Model also performed reasonably well (RMSEP = 1.42 m), but under predicted significantly in some instances (Figure 5.27).

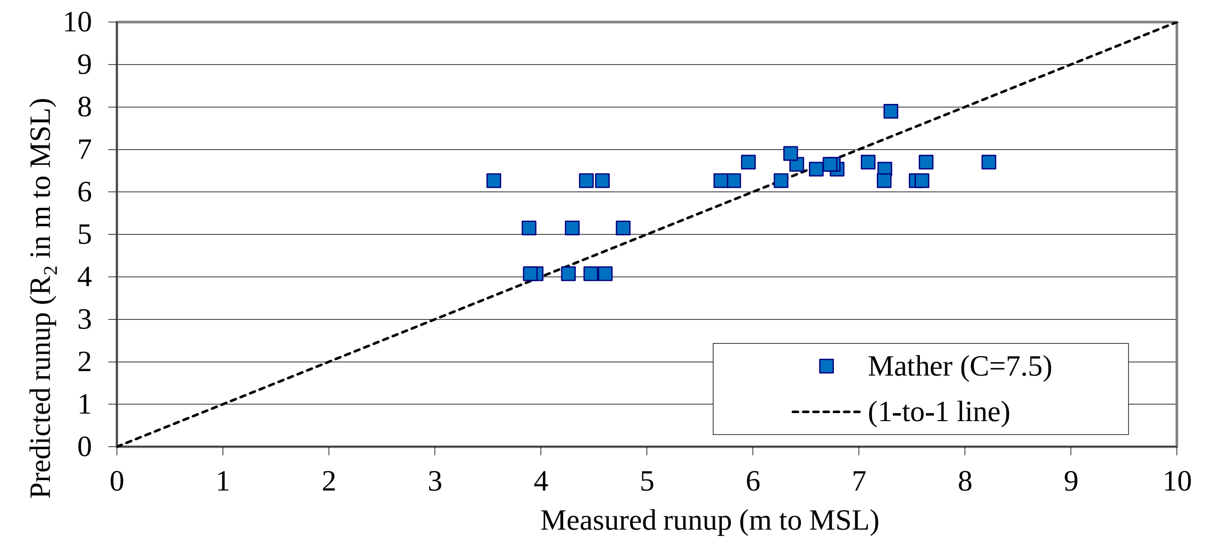


Figure 5.28: Predicted versus measured wave runup elevation (KZN) – Mather et al (2011) Model.

The Mather *et al* (2011) Model performed well against this data set (Figure 5.28) with a RMSEP of only 0.96 m.

Discussion and recommendations for application of the wave runup prediction methods

In terms of the objective Root Mean Square Error of Prediction (RMSEP) values, the Nielsen and Hanslow (1991) model (with H_0' as input) and Mather *et al* (2011) Model are respectively the best (RMSEP = 0.67 m) and third best (RMSEP = 1.16 m) of the models tested against the Table Bay data. Against the KZN data, the Nielsen and Hanslow (1991) and Mather *et al* (2011) models are respectively the second best (RMSEP = 1.32 m) and best (RMSEP = 0.96 m). While the Ruggiero *et al* (2001) Model did not perform quite as well against the KZN data (in fifth place, with a RMSEP of 1.77 m), it faired second best against the Table Bay data (RMSEP of only 0.68 m), and best of all the models against the West Coast data (very low RMSEP of only 0.24 m). While the Stockdon *et al* (2006) Model arguably still performed acceptably well (RMSEP = 1.18 m to 1.42), it faired less well than the aforementioned three models. The Diaz-Sanchez *et al* (2013) Model did not perform as well (RMSEP = 1.45 m to 3.01 m). Therefore, based on all of the foregoing tests from diverse coastal areas and a wide variety of local conditions, it is concluded that the three models of Nielsen and Hanslow (1991), Ruggiero *et al* (2001) and Mather *et al* (2011) are the best of the available models for application in South Africa . To further elucidate the performance of these three models, they were tested against a combined data set consisting of the three previously discussed data sets (West Coast, KwaZulu-Natal and Table Bay). The results are graphically illustrated in Figures 5.29 (Nielsen and Hanslow (1991) Model), 5.30 (Ruggiero *et al* (2001) Model) and 5.31 (Mather *et al* (2011) Model).

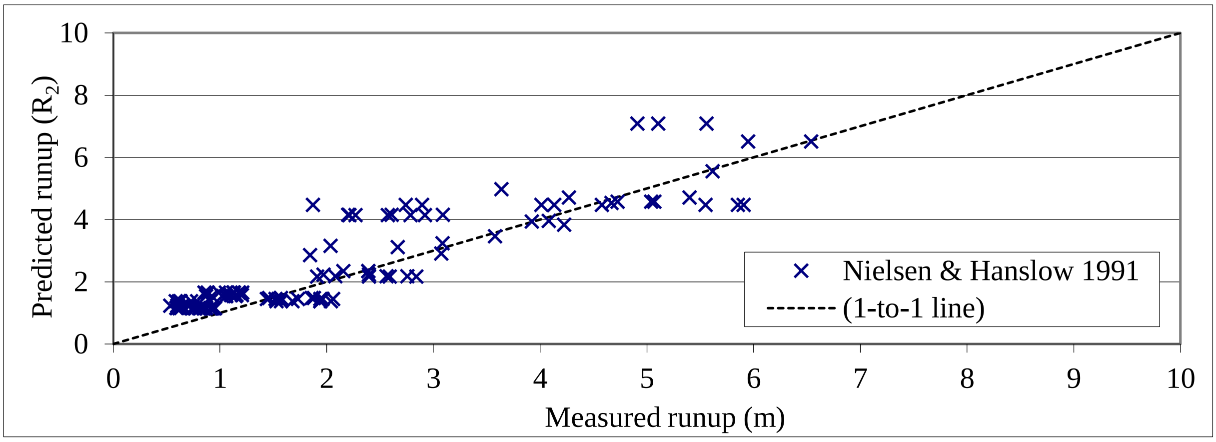


Figure 5.29: Performance of Nielsen and Hanslow (1991) Model against combined West Coast, KwaZulu-Natal and Table Bay data set.

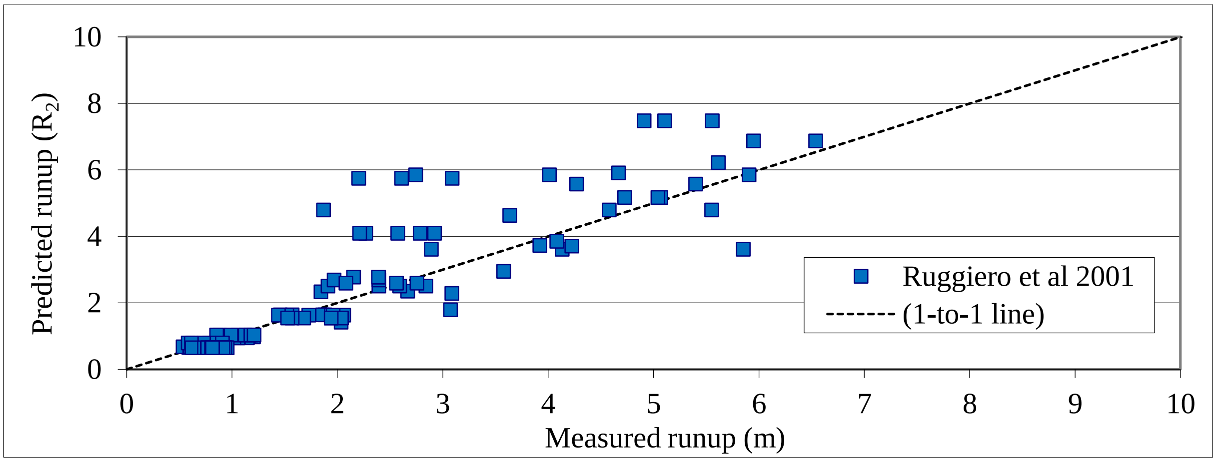


Figure 5.30: Performance of Ruggiero et al (2001) Model against combined West Coast, KwaZulu-Natal and Table Bay data set.

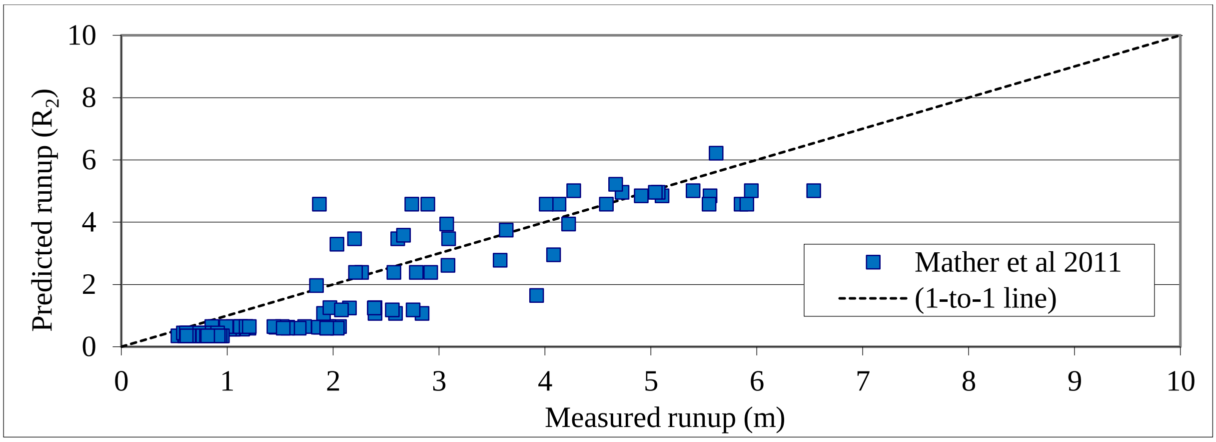


Figure 5.31: Performance of Mather et al (2011) Model against combined West Coast, KwaZulu-Natal and Table Bay data set.

In terms of performance with all three data sets (West Coast, KwaZulu-Natal and Table Bay), the Nielsen and Hanslow (1991) Model was the most consistent (Figure 5.29), had the lowest overall RMSEP (0.78 m) for the combined data set, and is therefore considered to be generally the most suitable. However, it should be used with certain adaptations as recommended here: the best results will be obtained with significant wave heights determined at about 20 m depth or less and then "reverse shoaled" to give the equivalent deep-water wave heights as input. Using methods such as those described in US Army, Corps of Engineers (1984), the shoaling coefficients from deep-water to for example 15 m depth with wave periods of 12 s and 16 s are calculated to be 0.98 and 1.08 respectively. (Bottom friction is not included in these simplified calculations.) To apply the Nielsen and Hanslow Model thus requires that the wave height be determined at 20 m depth or less (ideally recorded or modelled by means of a proper wave model such as SWAN, Booij *et al* 1999) and then reverse shoaled to derive the equivalent deepwater wave height, as well as the input wave period (T_p), the beach face slope (tan α), and the still water level. As mentioned in Section 5.3.1., and indeed illustrated in Figure 5.18, the Nielsen and Hanslow Model performs less satisfactory where the beach slope (tan α) is ≤ 0.06 (i.e. if 'flat' or low-gradient beach slopes or highly dissipative conditions exist). By calibrating the model against all the cases where the beach slope (tan α) is ≤ 0.06, it was found that the performance of the model could be improved significantly (RMSEP improved to 0.47 m) by changing the coefficient in Nielsen and Hanslow's (1991) formulation to a new proposed value of 0.04. The new formulation for the wave runup height (R_2), where tan(α) is less than or equal to 0.06, is thus as follows:

$$R_2 = WL + 1.98 \cdot (0.04 \cdot \sqrt{\beta})$$

(with the parameters as defined before)

The fit of the model predictions with the adapted Nielsen and Hanslow Model for tan α ≤ 0.06, against the measured data is illustrated in Figure 5.32.

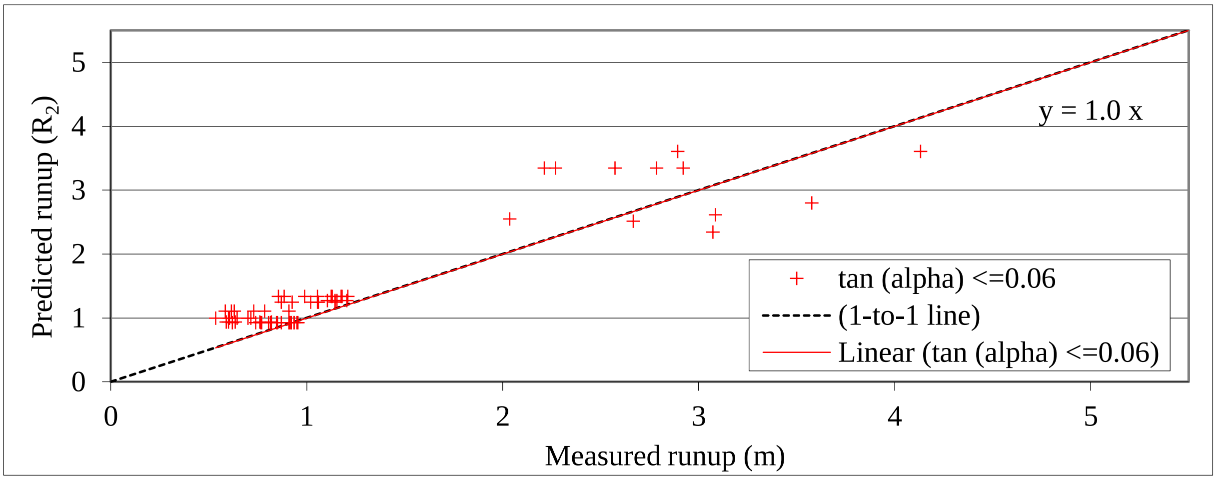


Figure 5.32: Fit of model predictions against measured data with adapted Nielsen and Hanslow Model for tan α ≤ 0.06.

These adaptations to the Nielsen and Hanslow model (i.e. the new coefficient for beach slopes ≤ 0.06, and the "reverse shoaled" wave input from shallower depths) broadens the applicability of the model to both mild and steep sloped beaches (i.e. dissipative and reflective) thus catering for both the lower energy areas inside some South African bays (e.g. St Helena, Table Bay, False Bay, Algoa Bay, etc.) and for the exposed high energy coasts (e.g. KZN, Cape south and west coasts, etc.). Thus, the Nielsen and Hanslow model can in this manner be applied to sandy shores within all five of the South African coastal regions (as characterised in Section 2.2.4).

The Ruggiero *et al* (2001) Model performed reasonably well against the combined data set (Figure 5.30), but had the highest overall RMSEP (0.91 m) of the three models, and is therefore considered to be generally the least suitable of these three. This model, however, clearly did perform well in conditions with low-gradient beach slopes or highly dissipative conditions from both the Table Bay (Figure 5.21) and West Coast (Figure 5.15) data sets. It is therefore provisionally recommended that the Ruggiero *et al* (2001) Model can be applied as an alternative to the Nielsen and Hanslow (1991) Model when the beach face slope (tan α) is ≤ 0.06. Such mild sloped beaches (i.e. dissipative) typically occur in the lower energy areas inside some South African bays (e.g. St Helena Bay, Table Bay, False Bay, Algoa Bay, etc.). Thus, the Ruggiero *et al* (2001) Model can be applied to the more sheltered (mildly sloped or dissipative) sandy shores within the South African coastal regions (as characterised in Section 2.2.4).

The Mather *et al* (2011) Model had the second lowest overall RMSEP (0.85 m) of the three models when applied against the combined data set (Figure 5.31). The RMSEP value could not be significantly improved by direct recalibration of the ”C” coefficient in the model. Where only “deep-water” wave heights are known (recorded in depths of about 50 m or more) or where data is lacking on the beach slope, the Mather *et al* (2011) Model can be applied. However, it is more applicable to exposed open coast locations (with steeper coastal slopes and more reflective conditions), such as are typical along the KZN coast, but also occur along portions of all five of the South African coastal regions (as characterised in Section 2.2.4). The distance to the 15 m contour is readily available from SAN bathymetry charts. The value of coefficient C should be set at 7.5 in open coast locations and even in semi-exposed locations, except if site specific wave runup calibration data is available (which could dictate either lower or higher C values). In well sheltered locations (e.g. deep inside bays or behind large headlands), the value of coefficient C should provisionally be set at 5 (even though Mather *et al* (2011) suggest a value of 4 in such conditions). These alternative values for coefficient C for application in semi-exposed locations and inside bays, serve to improve the applicability of the model to the full range of conditions typically found in the South African coastal regions (as characterised in Section 2.2.4). However, this recommendation (for setting C at 5 deep inside bays or behind large headlands) requires further validation based of field data. New research by Roux (2015) potentially provides some additional guidance on selecting appropriate values for coefficient C. Roux (2015) correlated this coefficient to Iribarren numbers and found that ”C” values between 3.0~5.0 are appropriate for low Iribarren conditions (0.25-0.4), while for higher Iribarren conditions of 0.75 to 0.8, ”C” values between 7.0~10 are appropriate. However, Roux’s recommendation is based on too limited data and therefore requires further validation.

Although the research conducted here favours the Nielsen and Hanslow model in most instances, it cannot be firmly concluded that this model is indeed preferred when the beach slope (tan α) is ≤ 0.06 (i.e. if ‘flat’ or low-gradient beach slopes or highly dissipative conditions exist). Additional field data is required to confirm unambiguiously which of the three models would be preferred under such conditions. In general, it can further be said that while both the Nielsen and Hanslow and Ruggiero *et al* models have been widely applied internationally (with at least reasonable success), the general validity and applicability for South African conditions (and ideally even broader) of the Mather *et al* (2011) model should ideally be investigated further. This would also assist in providing better guidance on the selection of coefficient C from its current wide range. Nevertheless, the results obtained with the Mather *et al* model (with adapted coefficients as recommended in the foregoing) are considered surprisingly good if the relatively few parameters included in the formulation are kept in mind. The Ruggiero model was also found to be a good alternative for the dissipative areas.

Regarding the Mather model, an alternative value for coefficient C is suggested for application inside the bays, to similarly improve the applicability of the model to the full range of South African conditions.

5.3.3. Application of prediction methodology for wave runup – case studies

Illustrative wave runup predictions for the South African coast

Having found the Nielsen and Hanslow (1991) model to be sufficiently valid and applicable to local conditions, the same methodology was applied to calculate the wave runup at various open-coast (exposed) locations around the South African coast, from the north-west coast (Port Nolloth) to the north-east coast (Richards Bay). Note, that these simulations are not applicable to sheltered locations within bays or behind headlands, where significant wave refraction would occur. In these sheltered locations, direct application of the Nielsen and Hanslow model with open coast deep-water wave heights will give rise to over-predictions as demonstrated for the Table Bay case in Section 5.3.2.

A range of plausible/realistic scenarios was selected, which included a number of assumed conditions common to all of the scenarios. In South Africa spring tides occur every two weeks, which means that the chances of storm waves coinciding with spring high tides are relatively high. Therefore, the input water level was set at spring high in the runup modelling. Two general beach slope categories were selected, namely mild slopes (1 in 18, ca 0.056) and steep slopes (1 in 9, ca 0.11) typical of the South African coast. Thus, wave runup elevations were modelled for various open-coast (exposed) locations along the South African coast, in conjunction with offshore wave heights as determined from NCEP (NCEP 2013) derived wave conditions along the South African coast (e.g. Rossouw and Theron, 2012 and for peak wave periods of 16 s). [A wave analysis was previously undertaken by Rossouw (Rossouw and Theron, 2012) of the offshore wave climate using NCEP hind cast wave data, from the NOAA/NCEP WAVEWATCH III Global Model (Tolman *et al* 2002), at deep sea offshore locations around the South African coast. The hindcast data sets contain about sixteen years of three-hourly wave and wind parameters.] The possible effects of climate change were then also included by assuming that the wave heights could increase by 10% due to stronger winds over the ocean (caused by climate change effects). Another effect that was considered was SLR, and for this parameter, values of 0.35 m (2050 best estimate) and 1 m (within 2100 best estimate range of 0.85 m to 1 m, Section 5.2.4) were selected; the two climate change effects of wave height increase and SLR were also combined.

Specific scenario combinations were then considered, as follows:

Figure 5.33 - Spring high tide together with the 1-in-10-year wave height:

The six lines depict, in increasing order of wave runup elevation, the present-day wave runup levels for both of the two slopes mentioned before, together with the present-day 1-in-10-year wave heights; future runup levels for a steep slope with a 10% increase in wave height; a steep slope with a 0.35 m SLR (i.e. 2050 scenario); a steep slope with a 0.35 m SLR plus a 10% increase in wave height; and a steep slope with a 1 m SLR (i.e. 2100 scenario).

The strong effect of the beach slope is obvious in the large increase in predicted wave runup elevation between the mild slope (bottom line) and the steep slope (second from the bottom line). This clearly illustrates the effect that local conditions would have on the extreme runup elevations actually experienced at a specific site. It is also interesting to note that the effect of a 10% increase in wave height is virtually the same as a 0.35 m increase in sea level. As expected, the western and southern Cape regions are subject to potentially the highest runup elevations, due to the more severe offshore wave climate. However, it should be kept in mind that this only holds true for a constant beach slope and complete wave exposure (open coast) around the South African coast (i.e. similar site characteristics). If, conversely, a similar offshore wave height were to be experienced at, for example, Cape Town and Durban, for a typical steep KZN profile slope the extreme wave runup elevation attained would then be much higher than for a typical flat or mildly sloped Cape beach. (This is besides other local factors that would affect the site-specific wave runup elevation.)

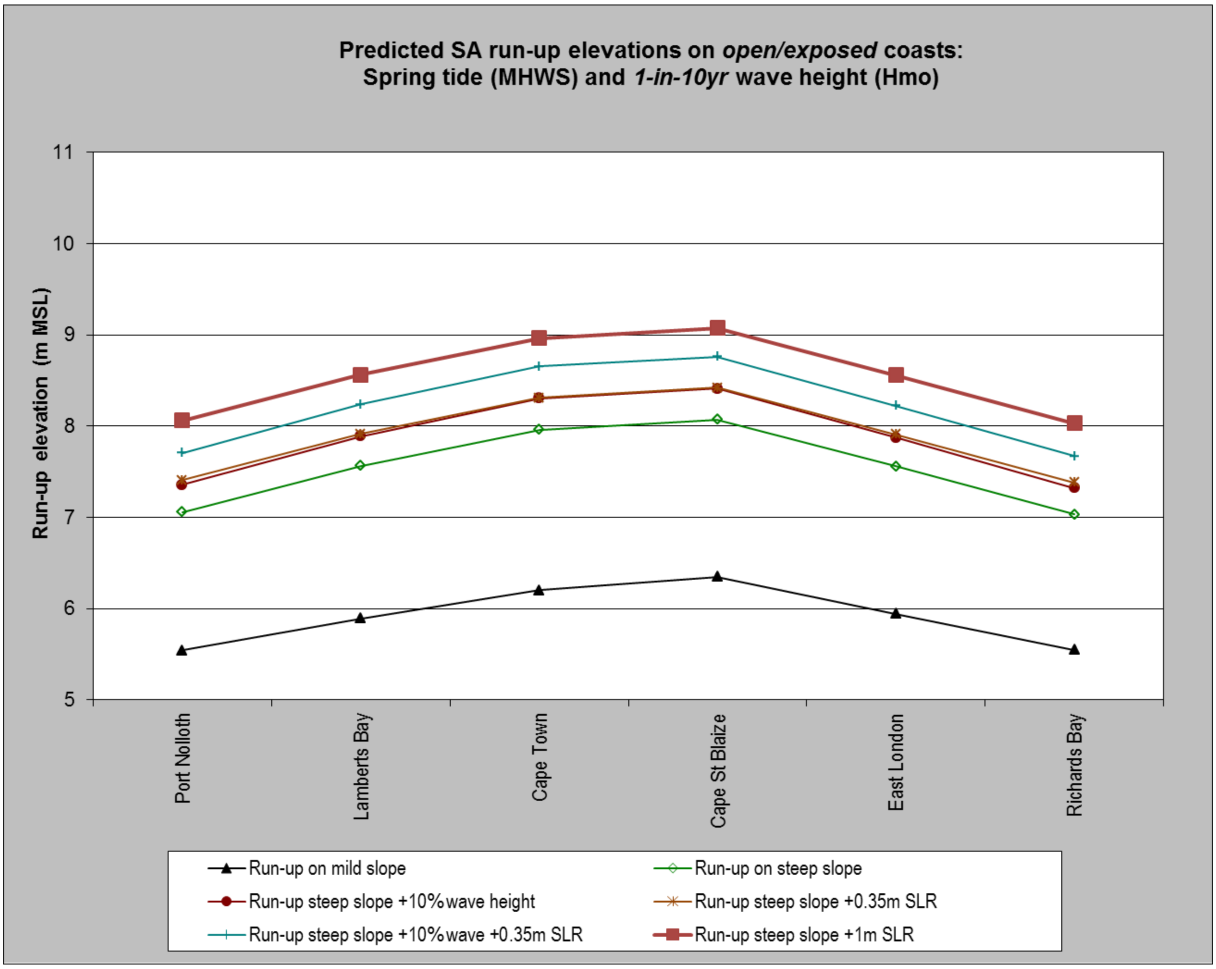


Figure 5.33: General runup levels predicted for different locations along the South African coast for 1-in-10-year wave height, including the potential effects of climate change (higher waves and/or sea level rise) *[The runup will be lower in sheltered locations within bays or behind headlands.]*

Figure 5.34 - Spring high tide together with the 1-in-50-year wave height:

Similar to the previous scenarios depicted in Figure 5.33, the six lines depict, in increasing order of wave runup elevation, the present-day wave runup levels for both of the two slopes but in this case, together with the present-day 1-in-50-year wave heights; future runup levels for a steep slope with a 10% increase in wave height; a steep slope with a 0.35 m SLR (i.e. 2050 scenario); a steep slope with a 0.35 m SLR plus a 10% increase in wave height; and a steep slope with a 1 m SLR (i.e. 2100 scenario). All of the same trends are observed, except obviously that the values of the runup elevations are all a bit higher for the 1-in-50-year wave heights compared to the 1-in-10-year scenarios. Note that the true joint probability of a 1-in-50-year wave height occurring in conjunction with spring high tide is actually less than just a 1-in-50-year wave event; in other words, the return period of the joint event would statistically be greater than once in 50 years (i.e. more rare or extreme).

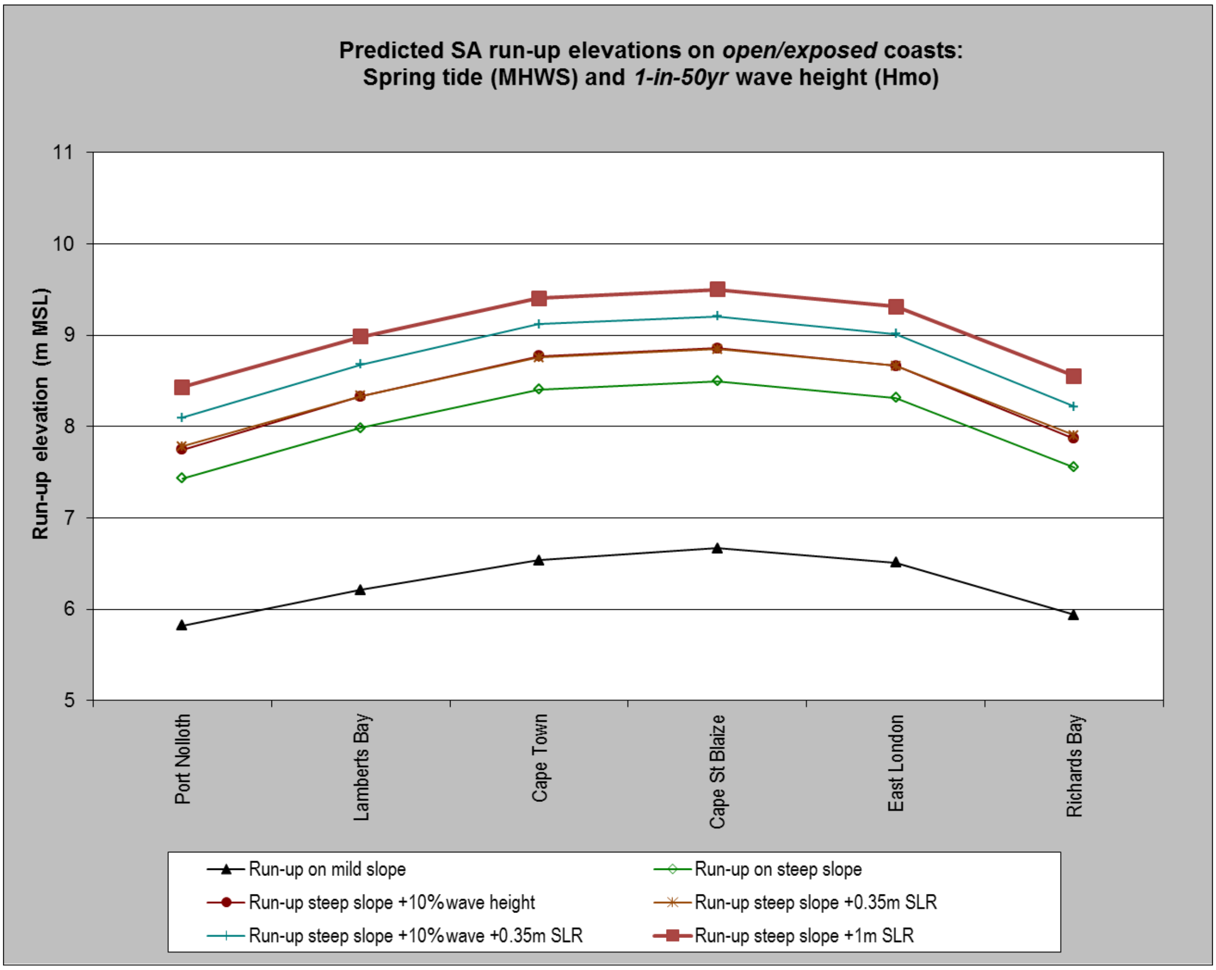


Figure 5.34: General runup levels predicted along the South African coast for 1-in-50-year wave height during spring high tides *[The runup will be lower in sheltered locations within bays or behind headlands.]*

<u>Figure 5.35</u> - Spring high tide + 1-in-10-year sea level residuals, together with the 1-in-10-year wave runup:

These scenarios are also similar to the scenarios depicted in Figure 5.33, namely spring high tide together with the 1-in-10-year wave runup. However, in this case, they also include inshore sea level increases not related to wave setup, but due to other effects (mainly hydrostatic and limited wind effects). Thus, 1-in-10-year sea level residuals as determined for each coastal region in Section 5.2.1 (Table 5.2) are also added to the input sea level. The future scenarios are also as before but now include the 1-in-10-year sea level residuals. The observed trends are relatively similar to before, except that the values of the runup elevations are significantly increased due to the extra sea level increase from the residuals (as expected). The true joint probability of a 1-in-10-year wave height occurring in conjunction with spring high tide and additional sea level increase due to low barometric pressure is again less than a 1-in-10-year wave event alone (in other words, the return period of the joint event is greater than once in 10 years, and is more rare/extreme). However, this is a quite likely scenario, as spring tides occur often (every two weeks) and low-pressure systems are often the cause of major sea storms. Thus, these factors have indeed occurred simultaneously in the past, leading to very severe conditions and impacts along numerous parts of the South African coast.

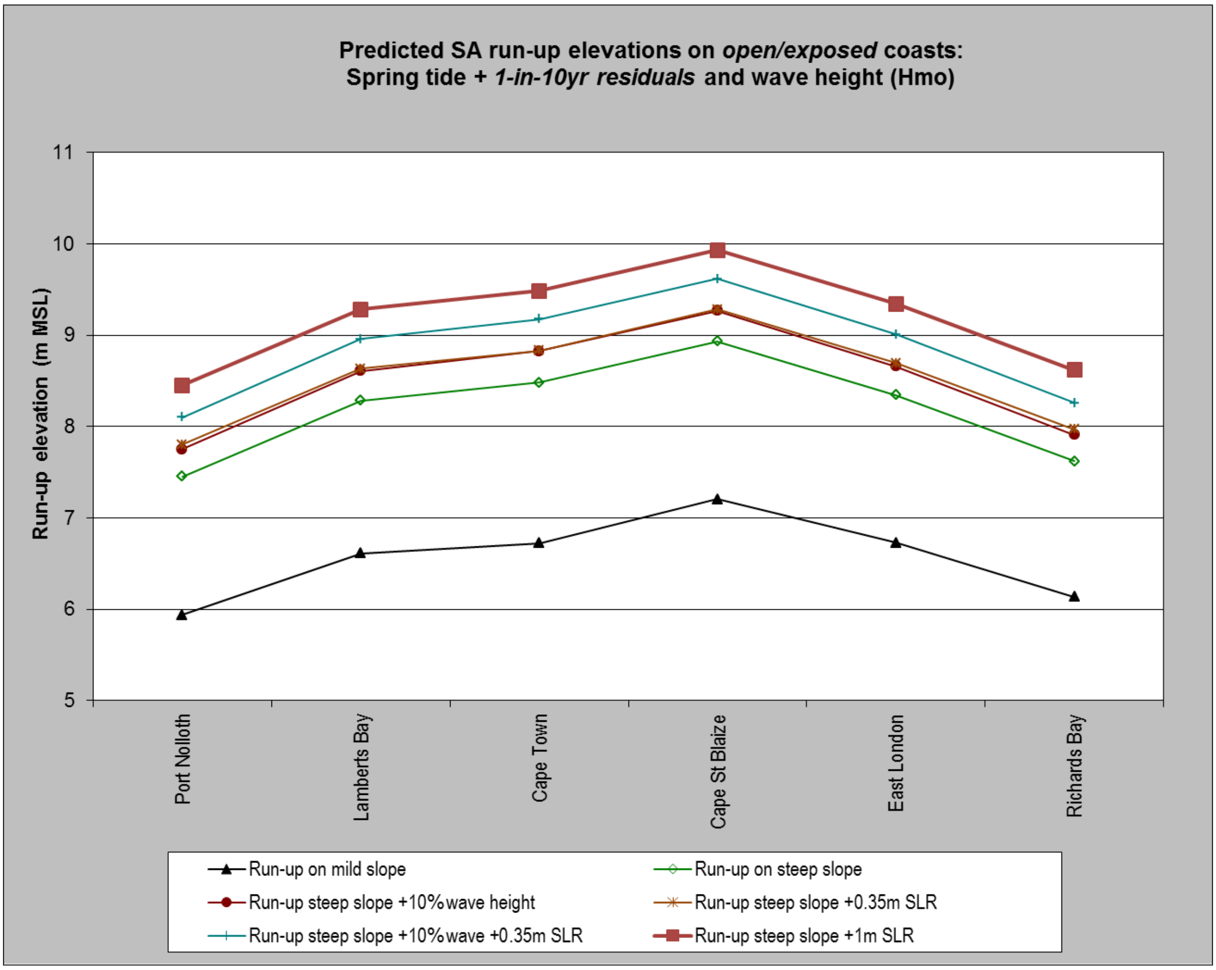


Figure 5.35: General coastal flooding levels predicted along the South African coast for both 1-in-10-year sea level residuals and wave runup during spring high tides *[The runup will be lower in sheltered locations within bays or behind headlands.]*

<u>Figure 5.36</u> - Spring high tide + 1-in-50-year sea level residuals, together with the 1-in-50-year wave runup:

These scenarios are similar to the scenarios depicted in Figure 5.35 but for the 1-in-50-year wave height as well as the 1-in-50-year sea level residuals as determined for each coastal region in Section 5.2.1 (Table 5.2). The same trends are observed, except obviously that the values of the runup elevations are all a bit higher for the 1-in-50-year scenarios. As before, the true joint probability of wave height in conjunction with spring high tide and additional sea level increase due to low barometric pressure is less (in other words, more rare/extreme than a 1-in-50-year wave height alone). Therefore, this is a relatively extreme (> 50-year return period), but quite plausible scenario, as discussed before. This could be considered to be a relatively severe (but still not the most extreme possible) scenario suitable or relevant for various planning purposes.

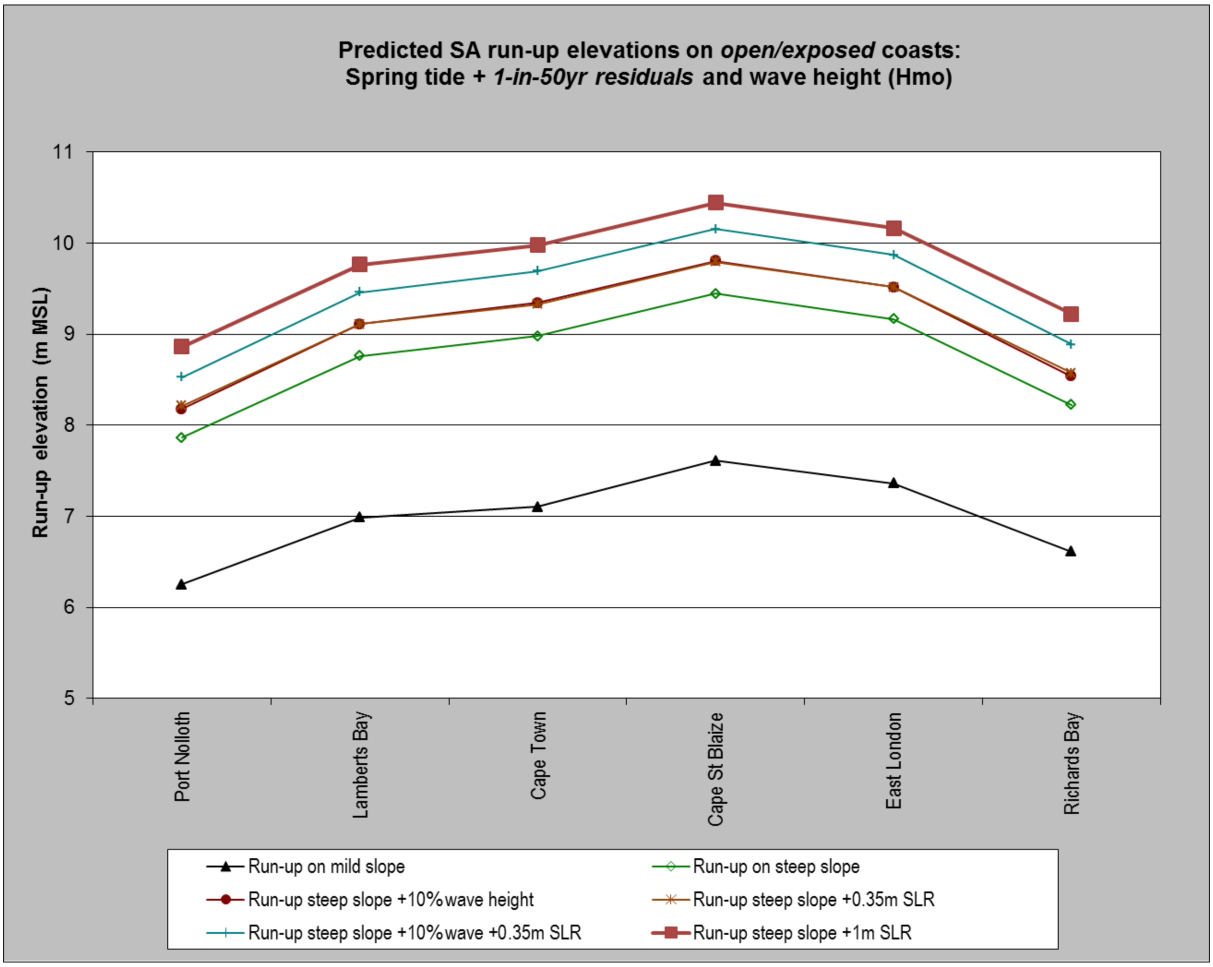


Figure 5.36: General coastal flooding elevations predicted along the South African coast for both 1-in-50-year sea level residuals and wave runup during spring high tides *[The runup will be lower in sheltered locations within bays or behind headlands.]*

General discussion on the illustrative wave runup predictions for the South African coast

Based on the foregoing, it is clear that the methodology that was applied to calculate the wave runup at locations around the South African coast, can indeed be efficiently applied in large study areas.

The foregoing scenarios and predicted wave runup elevations are illustrative values that may be applicable to various open coast locations along the South African coastline, depending on the specific circumstances. The values should not be uniformly applied, as site characteristics (e.g. slope) and local conditions (e.g. wave exposure/sheltering) will affect the actual in-situ wave runup.

A case in point is the wave runup elevations observed during the March 2007 storm along the KZN coast versus the runups during the September 2008 storm along the Cape coast. Wave heights off Richards Bay reached 8.5 m (Hs) during the 2007 storm (CSIR data) while wave heights of up to 10.7 m (Hs) were recorded off Cape Point during the 2008 storm (CSIR data). These values cannot be directly compared due to the differences in recording depths, but it can be said that the offshore conditions during the Cape storm were more severe than during the KZN storm. Yet, despite the higher offshore wave heights off the Cape coast, extreme runup elevations observed along the KZN coast were higher than those observed in the Cape. This can be ascribed to a number of reasons, two of the most important being related to the respective profile slopes and the wave exposure (/sheltering) found at the observation sites. Typical inshore-dune slopes (i.e. from the outer surf zone to the upper beach/dune) are much steeper along many KZN locations than along many Cape beaches. The steeper slopes result in much higher runup than on mild slopes, as has already been demonstrated in Figures 5.33 to 5.36. Furthermore, some of the Cape observations were made along typical embayed shorelines, whereas the extreme KZN observations were made along open exposed areas (no wave sheltering). Within bays, the waves are refracted and the wave energy is spread along a much longer shoreline. Within the most sheltered parts of deep bays, the waves impinging on the beach are much reduced compared to adjacent open-coast areas. Therefore, those two phenomena (besides some potential other effects) already account for the significant differences in the observed runup elevations. The severity of the impacts to the KZN coastal infrastructure and developments was also much greater than along the Cape Coast. However, this is also partially due to other important factors, such as the location (height or distance from the high-water line) of infrastructure and housing as well as the relative density of the structures along the shoreline. Along some parts of the KZN coast, more structures appear too close to the sea compared to general Cape coastal areas.

Potentially, in the worst-case scenario (most extreme case), all of the factors considered in Section 5.2, namely spring tide, hydrostatic, wind and wave setup, SLR and extreme wave conditions, could occur simultaneously. In a very special case and location this could possibly happen, but actually almost never would. For example, the highest wave runup elevations occur on open, exposed shorelines. However, extreme wind setup tends to occur in bays (where "piled up" water cannot easily "escape"), and much lower wind setup is observed along open coasts. Thus, these two phenomena, for example, virtually never attain their respective most extreme values in the same location. Therefore, scenarios combining all of these components were considered overly conservative and not appropriate for the purposes of this investigation. Similarly, it should be noted that within estuaries and harbours or very sheltered areas (e.g. deep into the lee of significant headlands or capes, behind islands) the wave runup phenomenon is mostly severely reduced (similar to wave setup). Nevertheless, where appropriate (e.g. detailed design of important coastal structures), high-resolution, fine-scale, site-specific investigations should properly consider the applicable values and probabilities of all of these potential components and events.

It can be said that the wave data on which the scenarios of the previous section are based contains virtually no cyclone wave events. However, in Section 5.2.2 it was mentioned that cyclone occurrence statistics in the Mozambique offshore region (ca. only two cyclones per year enter the Mozambique channel) showed an occurrence of about one third less in southern Mozambique relative to central Mozambique (Theron *et al*, 2012). Also, very few cyclones approach even farther south close to the South African coast (in the order of one per decade in northern KZN), and even when they do, they have usually lost much of their strength by the time that they enter South African waters. Thus, potential setup/surge or wave runup due to cyclones, which exceed the effects of the wave scenarios already included earlier (in the illustrative wave runup predictions for the South African coast), is extremely unlikely and deemed overly conservative. Therefore, specific focus on surge or wave runup due to cyclones is not included in this study (but could be included in further studies where this may be more relevant, for example in Mozambique).

5.3.4. Application of prediction methodology – Further case studies illustrating climate change effects on wave runup

The same methodology (Nielsen and Hanslow model) was again applied to further investigate the impact of SLR on runup return periods and occurrences. One of the impacts of SLR is that waves will reach farther inland than at present, which implies that present coastal development setback lines (of which few exist) have to be adapted. Factors that codetermine the location of setback lines are storm wave runup elevations and how far the shoreline will retreat due to erosion, which are in turn affected by the amount of SLR that is expected and the projected increases in storminess. Therefore, realistic scenarios of SLR and potential increases in wave heights were combined with calculations to determine the resulting effects on wave runup. An additional objective of the KZN and Table Bay case studies, is to further demonstrate that the applied methodology can indeed be efficiently applied in large study areas.

Application of prediction methodology for extreme water levels, sea storms and wave runup – Durban, KZN case study:

To clearly illustrate the strong effect that SLR has, a low SLR value was first applied. The mean value of the IPCC Fourth Assessment Report SLR predictions is about 0.4 m by 2100 (IPCC 2007). Using this prediction of future sea levels, it was found that the same extreme wave runup elevations as occurred during the extreme 2007 KZN storm in South Africa would be reached by waves 10% lower (H_{m0}) than those recorded during the peak of the 2007 storm. This means that based on the calculated return period of the 2007 storm (and assuming that the statistical distribution of extreme waves remains about the same over the next 100 years), the return period for the same extreme runup heights is effectively halved. In other words, the probability of such extreme conditions occurring again is basically doubled, or statistically, such situations are likely to occur about twice as often over the long term for an SLR of only 0.4 m. (Note, that as discussed in the next paragraph, a SLR of 0.4 m by 2100 is below the range of scenarios considered to be most suitable for planning purposes in this thesis.)

In Section 5.2.4 it was concluded that the most appropriate scenario (or “central” estimate) of SLR by 2100 is about 0.85 m to 1 m (with a plausible worst-case scenario of 2 m and an appropriate low scenario estimate of 0.5 m). Therefore, in view of the newer SLR predictions (post IPCC 2007), the effects of a 1-m SLR (2100 scenario) on runup levels were also quantified. It was thus calculated that a wave height of 24% less than the 2007 KZN storm would result in similar runup elevations if sea level rose by 1 m. The results are alarming in that the return period of the 2007 event (in terms of high runup elevations) would effectively be subject to a six-fold reduction. In other words, the probability of such extreme events (in terms of high runup elevations) as those experienced during 2007 happening again would be six times greater, or statistically, such impacts are likely to occur six times as often in the long-term due to an SLR of 1 m. Thus, due to climate change effects, potential impacts similar to those experienced during the 2007 storm (Figure 5.37), could in future possibly occur much more frequently.

Figure 5.37: Example of the impact of the March 2007 KwaZulu-Natal sea storm *(Photo: D Phelp)*

Application of prediction methodology for extreme water levels, sea storms and wave runup – Table Bay case study

Coastal areas within Table Bay (Bloubergstrand to Melkbosstrand) near Cape Town (Figure 5.38) were selected to illustrate how such runup calculations (applying the Nielsen and Hanslow model) may be used to determine present and future vulnerable areas, which should be taken into consideration in locating local setback lines. The western and southern Cape were subject to significant storm impacts on 1 September 2008. Runup data was collected by the CSIR following the 2008 storm, and part of these runup lines is presented in Figure 5.38.

To map areas that are susceptible to wave runup requires coastal topographical data as input. This data was provided by the City of Cape Town, while the beach slopes that were used in the calculation of the runup levels for the two sites were obtained from beach profiles surveyed by the CSIR. As before, the range of most appropriate SLR scenarios for 2100 is given as 0.5 m to 2 m (Section 5.2.4). Accordingly, this is the SLR range selected for use in the example shown in Figure 5.38. The potential impacts of climate change on wave runup at these two sites are illustrated by calculating the runup levels with increased sea levels of 0.5 m, 1 m and 2 m. The most severe potential impact is, as is to be expected, observed when an SLR of 2 m and a 10% increase in the storm wave heights coincide.

The runup mapped for the 2008 storm does not indicate significant impacts in this area, which is borne out by observations during the storm. However, according to the predicted runup mapped in Figure 5.38, even a 1-in-20-year storm (without adding any SLR effects) will start causing problems for existing infrastructure and developments. As progressively higher sea levels are added and the scenarios become more severe (as they may well do over time), the predicted runup increases and the potentially vulnerable areas become increasingly larger. Clearly, once SLR exceeds about 1 m, a mere 1-in-20-year sea storm could cause major problems in the highly built-up areas near Blouberg. In addition, major transport infrastructure (the coastal trunk road) is also potentially at risk due to an increased sea level (Figure 5.38); all the more so were sea storm occurrence or severity also to increase. (Note, that the runup method applied assumes a single slope, while the local topography in some locations includes two alongshore dune ridges; having a dune crest followed by a trough followed by another crest, will also affect, and probably reduce the extent of the actual flooding.)

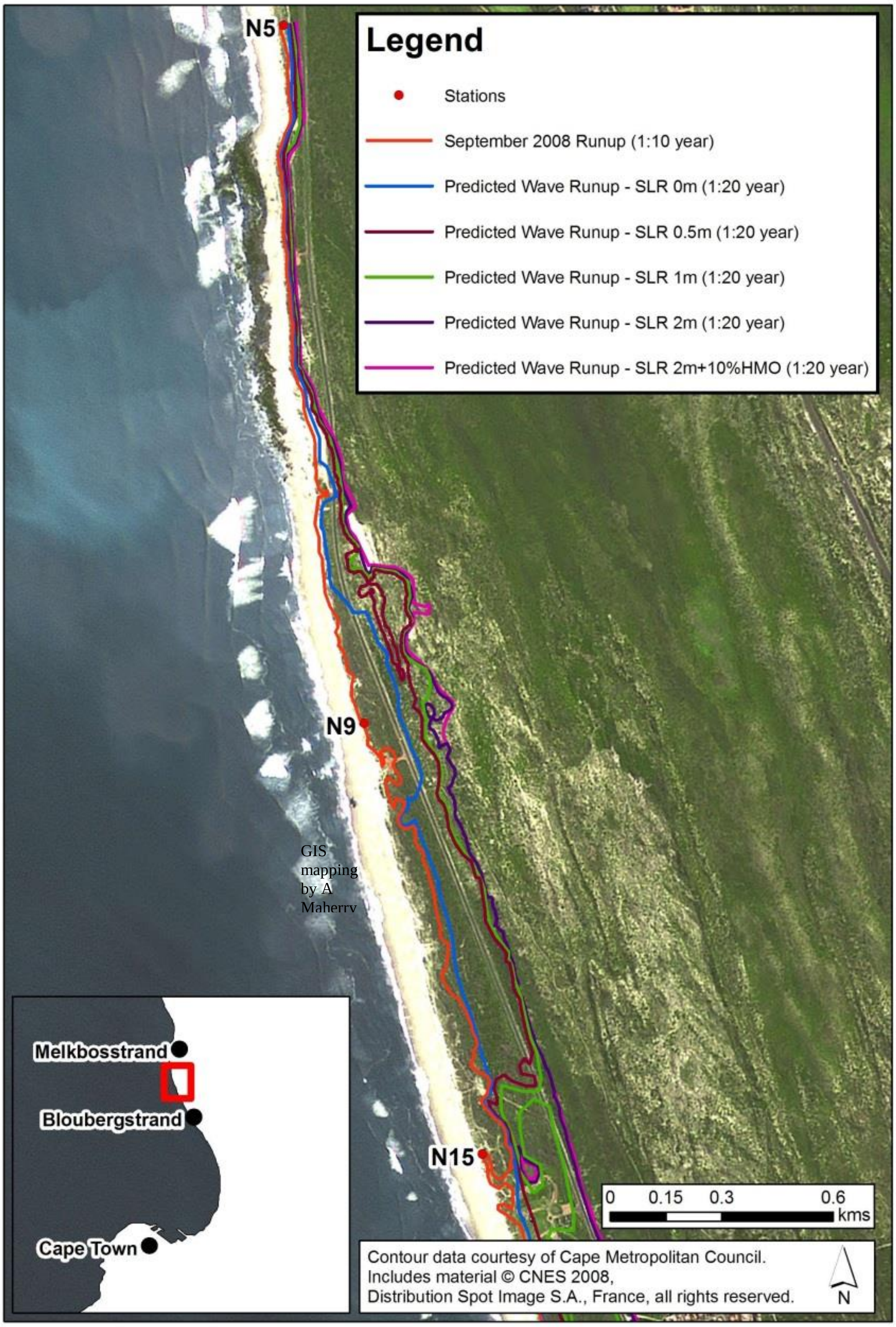


Figure 5.38: Illustration of the predicted effects of climate change on coastal runup lines near Blouberg.

Application of prediction methodology for extreme water levels, sea storms and wave runup – Mossel Bay case study

Using a relatively exposed shoreline location (Tergniet) as an example of a typical area prone to storm-waves within greater Mossel Bay, simulations of wave runup elevation (applying the Nielsen and Hanslow model) for spring high tide and south-south-westerly swell conditions are currently predicted to range between approximately 5.7 m for a 1:1 year return period to 6.5 m for a relatively extreme event with a 1:50 year return period (Figure 5.39). Potential future wave runup was modelled by assuming only a 0.5 m rise in sea-level (within the most appropriate range of SLR predictions from recent publications as per Section 5.2.4), and by applying a 6% increase to offshore extreme waves (based on regional projections from metocean climate modelling [Mori *et al*, 2010]). Under these future wave climate and sea-level rise predictions, the current 1:50 year wave runup elevation (at about 6.5 m MSL) is forecast to be reduced to about a 1:3 year return interval at this location.

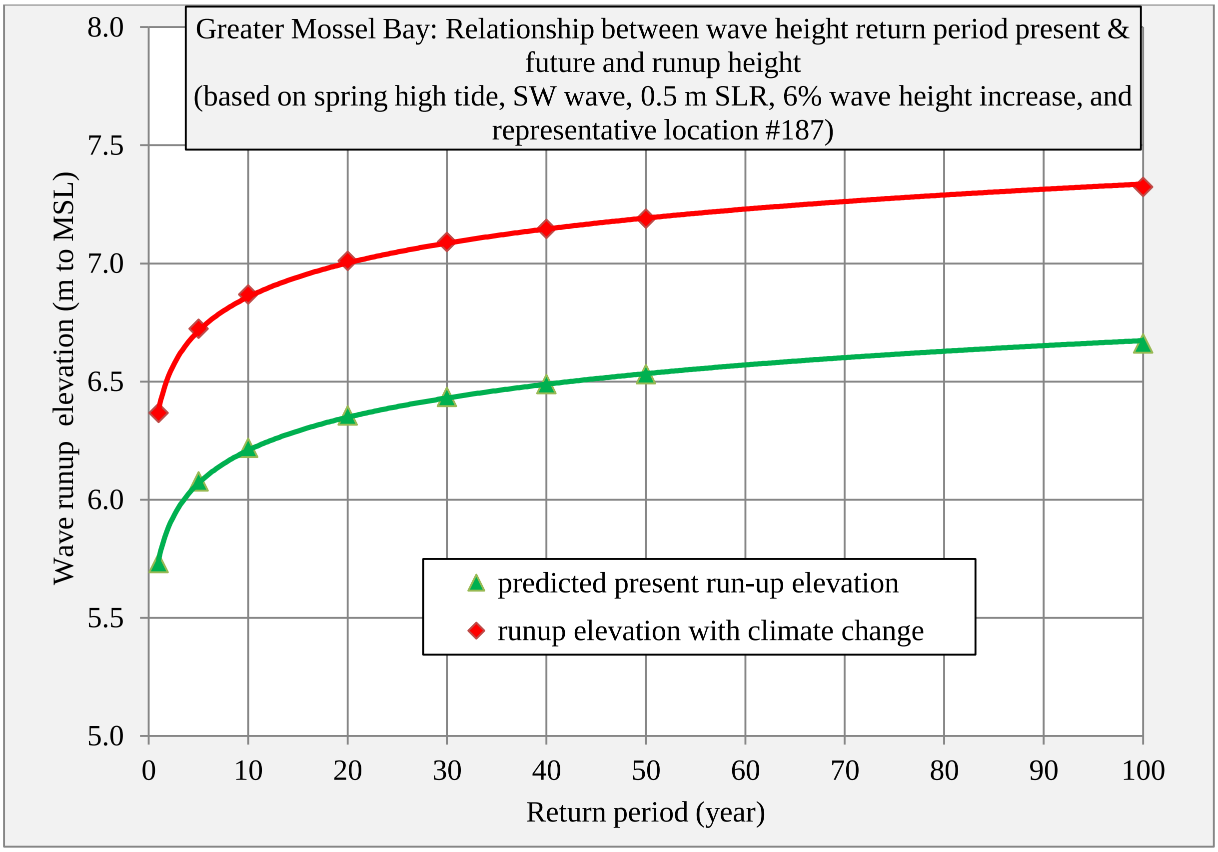


Figure 5.39: Wave runup elevations for various storm-wave return intervals for different scenarios of beach slope and climate change.

5.3.5. Runup on rocky shores

Where slopes of rocky shorelines are relatively steep (in many South African coastal areas slopes on rocky shorelines are much steeper than on the sandy shores), and the surface of the rocky area is relatively smooth, wave runup heights can be much greater than along sandy shorelines subject to similar offshore wave conditions. Many formulae and models have been developed to predict wave runup on revetments and rubble/rock slopes (e.g. Hughes, 2005). These have often been used in the past to predict runup on rocky slopes with modification factors for the surface (rock) roughness. One such example is the formula by Van der Meer (De Waal and van der Meer, 1992; Van der Meer and Stam, 1992) which was widely used in the past.

A more recent formula is given in CIRIA; CUR; CETMEF (2007 – "The Rock Manual") which has also seen wide application. This formula is based on a prediction curve developed for runup ($R_{u2\%}$) of waves on smooth slopes. According to the Rock Manual formula, the 2% runup height ($R_{u2\%}$) is a function of the deepwater wave height (H_{m0}) and the surf similarity parameter, which is also known as the Irribaren number (Irribaren, 1949; CIRIA; CUR; CETMEF, 2007). The basis of the Rock Manual formula is therefore similar to that of the Ruggiero *et al* (2001) and the Diaz-Sanchez *et al* (2013) models for runup on beaches, which models have been discussed in Section 5.3.2. However, the Rock Manual formula also incorporates various correction and safety factors, which are not included in these two beach runup models. A correction factor (γ_b) is, for example, included in the Rock Manual formula to account for the wave incident angle. This is appropriate, as wave incident angles on coastal structures (e.g. revetments) can be relatively large (due to factors such as greater water depth at the structure resulting in less wave refraction). Wave incident angles on beaches are typically very small in comparison and therefore a specific correction factor is not included in the beach runup formulae (although wave refraction and other wave modification factors may be accounted for by means of wave models to derive the input conditions for some of the beach runup formulae). The Rock Manual formula is given as follows (CIRIA; CUR; CETMEF, 2007):

$$R_{u2\%}/H_{m0} = A.\ \gamma_b\ .\ \gamma_f\ .\ \xi_m^{-1} \text{ for } 0.5 < \gamma_b\ \xi_m^{-1} < 8$$

where:

A = safety factor (= 1.65 for values without safety margins. The Rock Manual formula was developed for design of coastal structures (e.g. revetments). Thus a safety factor is included, which serves a different purpose than the beach runup prediction formulae discussed in the previous sections.),

γ_b = correction factor for wave incident angle,

γ_f = correction factor for rough slopes,

ξ_m = surf similarity parameter (also known as the Irribaren number).

For rough slopes, the runup reduction factor for roughness is given as:

Pitched stone: γ_f = 0.80–0.95

Armour stone – single layer on impermeable base: γ_f = 0.70

Armour stone – two layers on impermeable base: γ_f = 0.55.

Therefore, the runup reduction factor (γ_f) for roughness of rocky shores is taken to be between 0.55 to 0.95 depending on how rough the slope is. The “Rock Manual” can thus be used to estimate the runup along rocky shores.

Breetzke *et al* (2012) and Van Weele *et al* (2013) applied the method in the Eurotop manual (Pullen *et al*, 2008) to determine the wave runup heights on rocky shorelines along the South African West Coast and the Overberg coastal regions. The Eurotop manual is based on extensive research from laboratories and many field study areas and is newer than the “Rock Manual”, and is therefore the generally preferred method, and also suitable for wide application in South Africa in determining flooding levels for setback lines along rocky shores.

5.4. Conclusions

Extreme seawater levels, storm surge and wave runup prediction are all part of determining coastal flooding elevations, which is one of the two major abiotic components of setback lines, and also a major focus of this thesis. Significant drivers of high inshore seawater levels are tides, wind setup, hydrostatic setup, wave setup and, in future, sea-level rise, which all affect the still-water level at the shoreline. The additional significant component of extreme inshore seawater levels in the South African context is the wave runup. South African seawater level recordings and related information are discussed (Section 5.2), and the tidal ranges for the South African coast are summarized in Table 5.1. Extreme South African seawater levels excluding tides (thus mainly due to wind and inverse barometric setup) have been analysed for all of the South African stations (Theron *et al*, 2014), and the results (i.e. residuals for various return periods) as summarized in Table 5.2, are discussed.

Based on an extensive literature review (including the most recent findings up to date), it is concluded that the most appropriate (or 'central estimate') of SLR by 2100 is ~ 0.85 m to 1 m, with a plausible worst-case scenario of 2 m and a low estimate of 0.5 m. The corresponding best estimate (mid-scenario) projections for 2030 and 2050 are about 0.15 m and 0.35 m, respectively.

Extreme values are determined for realistic combinations of all the inshore seawater level components (described in Sections 5.2.1 to 5.2.4), as applicable to each South African coastal region. Based on these calculations and South African offshore wave conditions, estimates are made of the regional open coast storm surge levels around the South African coast for the main offshore wave conditions. This provides a robust first-order coarse storm surge level assessment for the South African coastal regions, which feeds into coastal flooding determinations (Section 5.3.3), and also indicates the relative open coast flooding levels of the different South African coastal regions.

In the foreword to this chapter, the strong requirement is stated for determining coastal flooding levels, including the important component of wave runup. It was therefore necessary to test and find suitable wave runup models. Following an extensive literature review and testing of 11 different wave runup models against local data from diverse coastal areas and a wide variety of local conditions, it is concluded that the three models of Nielsen and Hanslow (1991), Ruggiero *et al* (2001) and Mather *et al* (2011) are the best of the available models for application in South Africa. With an overall Root Mean Square Error of Prediction (RMSEP) of only 0.78 m, it is concluded that the model of Nielsen and Hanslow (1991) is generally the most suitable of the available models and is sufficiently robust with defendable results adequate for application in South Africa. It should however be used with certain adaptations as recommended here. The Ruggiero *et al* (2001) Model, however, clearly performed well with low-gradient beach slopes or highly dissipative conditions. It is therefore provisionally recommended that the Ruggiero *et al* (2001) Model be applied when the beach face slope ($\tan \alpha$) is ≤ 0.06. Such mild sloped beaches (i.e. dissipative) typically occur in the lower energy areas inside some South African bays (e.g. St Helena Bay, Table Bay, False Bay, Algoa Bay, etc.). Thus, the Ruggiero *et al* (2001) Model can be applied to the more sheltered (mildly sloped or dissipative) sandy shores within the South African coastal regions (as characterised in Section 2.2.4).

The determination of wave runup heights on rocky shorelines by means of the comprehensively researched and extensively tested method in the Eurotop manual (Pullen *et al*, 2008) is adequate for South African application and does not need further research for incorporation into determination of setback lines.

Where only deep-water heights are known, or where data is lacking on the beach slope, the Mather *et al* model can be applied for estimating runup on sandy shorelines. With a low overall RMSEP of 0.85 m from tests on diverse coastal areas and a wide variety of local conditions, it is sufficiently robust and yields defendable results for determination of flooding elevations as input into setback lines. However, it is more applicable to exposed open coast locations (with steeper coastal slopes and more reflective conditions), such as are typical along the KZN coast, but also occur along portions of all five of the South African coastal regions (as characterised in Section 2.2.4). The inputs required are the deep-water wave height, the distance to the 15 m contour and the still water level, which makes this model particularly suitable for very "data poor" environments. The value of coefficient C should be set at 7.5 in open coast locations and even in semi-exposed locations. In well sheltered locations, the value of coefficient C should provisionally be set at 5. These alternative values for coefficient C for application in semi-exposed locations and inside bays, serve to improve the applicability of the model to the full range of conditions typically found in the South African coastal regions (as characterised in Section 2.2.4). However, these recommendations (for setting C at 7.5 or 5 respectively) require further validation based of field data.

Generally, the Nielsen and Hanslow model is the most suitable for sandy shorelines; the best results will be obtained with significant wave heights determined at about 20 m depth or less and then "reverse shoaled" to give the equivalent deep-water wave heights as input. The other inputs required are the wave period, the beach slope, and the still water level. Thus, the input data requirements are still acceptable for "data poor" environments. The adaptations to the Nielsen and Hanslow model (i.e. the new coefficient for beach slopes ≤ 0.06, and the "reverse shoaled" wave input from shallower depths) broadens the applicability of the model to both mild and steep sloped beaches (i.e. dissipative and reflective), thus catering for both the lower energy areas inside some South African bays (e.g. St Helena Bay, Table Bay, False Bay, Algoa Bay, etc.) and for the exposed high energy coasts (e.g. KZN, Cape south and west coasts, etc.). Thus, the Nielsen and Hanslow model can in this manner be applied to sandy shores within all five of the South African coastal regions (as characterised in Section 2.2.4). The Nielsen and Hanslow wave runup model was applied in various case studies, yielding illustrative wave runup predictions for the South African coast. Further case studies applying the prediction methodology for extreme water levels, sea storms and wave runup, and illustrating climate change effects on wave runup, were conducted for Durban, Table Bay and Mossel Bay. The case studies also demonstrated that the applied methodologies can indeed be efficiently applied in large study areas.

Based on the foregoing it is concluded that appropriate methods have been found to determine coastal flooding levels, which can be applied in all the South African coastal regions in a "data poor" environment, and that can be efficiently applied in large study areas, but that still yield sufficiently robust and defendable results. In conjunction, recommendations have been made for appropriate, practical and implementable methodologies to determine the coastal flooding components of setback lines in South Africa.

Chapter 6: Shoreline changes and coastal erosion

Coastal erosion (both long- and short-term) is the second of the two major abiotic components of setback lines (the other major abiotic component is coastal flooding as addressed in Chapter 5). Thus, the research objectives of this chapter are to: find or derive appropriate methods to quantify shoreline changes and predict coastal erosion in a "data poor" environment, that can be efficiently applied in large study areas, but that are still sufficiently robust and defendable; and to make recommendations for appropriate, practical and implementable methodologies to determine the coastal erosion/recession components of setback lines in South Africa.

6.1 Introduction

Coastal erosion was the main focus of setback lines determined in South Africa up to 2008 and remains a key component of all setback lines. Many different approaches have been developed to quantify, simulate or predict erosion of sandy shorelines. In this chapter approaches are pursued that are more suited to poor data availability and can be efficiently applied in large coastal areas, to overcome the constraints associated with many conventional methods. Thus, new methods are developed and two alternative approaches are proposed to quantify shoreline erosion.

Shoreline 'stability' or the probability of erosion (and/or under-scouring of structures) is affected by many drivers, processes and activities, some of which are natural and some of which are due to anthropogenic actions. Most of these factors are listed and categorised in the following diagram (Figure 6.1).

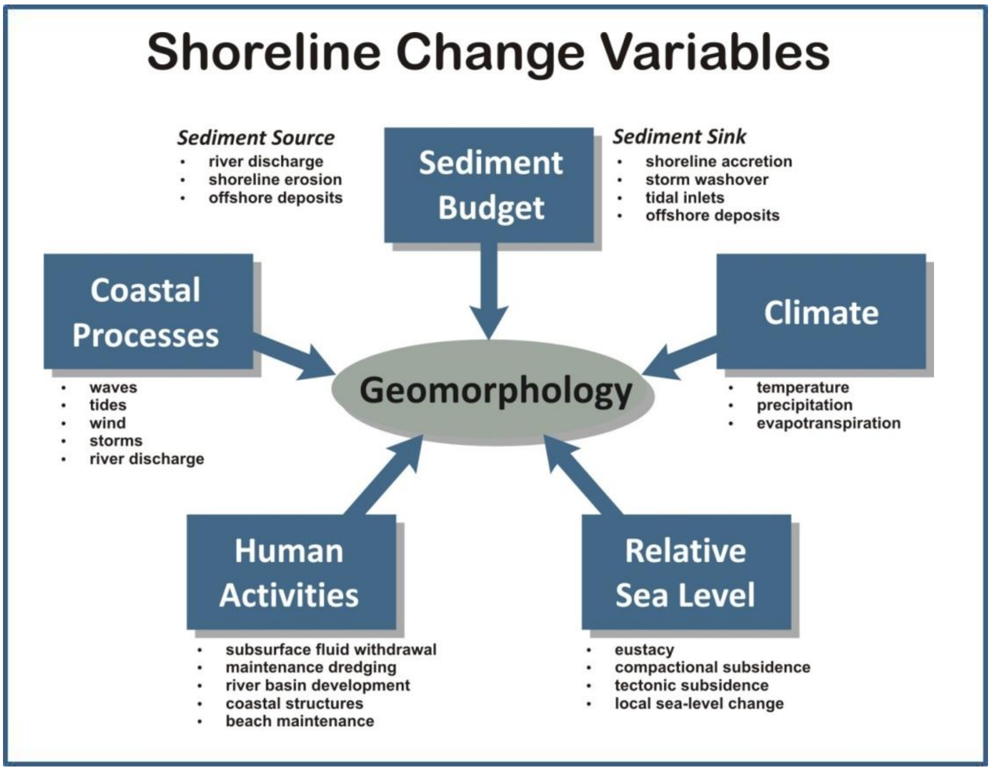


Figure 6.1: Drivers, processes and activities affecting shoreline "stability" or erosion

Shoreline changes (such as the location of the shoreline) can be differentiated into either short- or long-term changes. Short-term shoreline changes are regarded as the variations that occur on a temporal scale of typically hours to weeks. Such changes are often largely driven by wave action (especially sea storms), but are affected by most of the coastal processes listed in Figure 6.1 (including currents), as well as some of the other processes and activities listed in the figure.

Long-term shoreline changes are regarded as the changes and trends that occur on a temporal scale of typically years to centuries (or sometimes even longer). Such changes are often due to anthropogenic activities such as river basin development (e.g. dams or sand mining), breakwater construction, dredging of port entrance channels, sand bypassing at ports, etc., resulting in changes observed over years to usually in the order of about a century. A gradient in the longshore transport rate along a coastline can, in the long term, cause either erosion or accretion problems. These gradients can sometimes be identified by analysing long-term coastline changes (such as the analyses in Section 6.3). Other drivers such as SLR result in more gradual changes observed over periods of several decades to centuries and longer.

For a further discussion of causes/drivers, processes and time-scales regarding shoreline change, reference is made to the broad summary presented in Camfield and Morang, 1996, while Feagin *et al*, 2005 also discuss the causes of coastal erosion. Whilst suitable methods to quantify long-term shoreline changes and coastal recession are well established (as will be discussed in Section 6.3), this is arguably not the case for short-term changes and erosion predictions. Thus, the main focus of this Chapter is on short-term shoreline changes and erosion prediction.

6.2 Short-term shoreline changes and coastal erosion

6.2.1 Cross-shore transport/morphological models

Existing cross-shore sediment transport and morphology models

Current practise to quantify shoreline erosion often utilizes the application of numerical cross-shore sediment transport and/or morphology models. A review of the literature indicates that such models include: SBEACH (Larson and Kraus, 1989), Durosta (Steetzel 1987, 1993a), EDUNE (Kriebel, 1995), Vellinga (Vellinga 1982, 1986), DUNERULE (Van Rijn, 2008, 2009), CSHORE (Kobayashi *et al*, 2009), C2SHORE (Johnson and Grzegorzewski 2011; Grzegorzewski *et al* 2013), XBeach (Roelvink *et al* (2009), UNIBEST (Delft Hydraulics, 1994, 2005)) as well as others described by Nishi & Kraus (1996), Rakha *el al* (1997), Schoonees and Theron (1995), Swart (1974) and Stive *et al* (1996). In general, it can be said that the more modern (and complex) models are more process based and thus theoretically superior to the more empirical older models.

Of these modern models, the XBeach model (Roelvink *et al* (2009) is seeing increasing application, probably somewhat assisted by its being an open-source program (i.e. freeware). Based on its accessability, free use and process based approach, XBeach was therefore investigated further for potential application in determining erosion setbacks in the South African context. XBeach (for eXtreme Beach behavior) has been developed to model the nearshore response to hurricane impacts and storms (Roelvink *et al*, 2010). Thus, the main objective of the model was to test morphological concepts for the case of dune erosion, overwashing and breaching. The approach in XBeach is short-wave averaged, but long-wave resolving of waves, flow and morphology change in the time-domain, and regards swash and overwash motions, dune erosion, overwashing, breaching and full inundation of the profile. The intention was for the model to be driven by boundary conditions from surge and spectral wave models. According to Roelvink, it is not suitable for large-scale or long-term applications. This is in accordance with Bolle *et al* (2010), who state that the XBeach model can be applied to areas extending several kilometres in the longshore and about a kilometre in the cross-shore, and that this limited extent implies that it needs boundary conditions of tidal- and wind/pressure-driven water levels, deeper-water wave boundary conditions and bathymetry. By implication, the XBeach model is not suitable for determining coastal erosion on a large scale, but could for example still be employed to compare results with other methods in a small study area. According to Van Rooijen (2011), morphodynamic process-based numerical models tend to overestimate the seaward directed sediment transport in the swash zone, especially for mild conditions. He also concluded that the XBeach surf beat approach most likely overpredicts (offshore) sediment transport rates in the swash zone for more reflective beaches. Vousdoukas *et al* (2011), showed that alongshore profile morphology variations even for the same site, required different calibration settings in XBeach. They found that model sensitivity to calibration settings appeared to increase with beach slope. Other findings were that the model overestimated berm erosion and avalanching/beach scarp formation (compared to the study site). Their study highlights that predicting beach profile morphodynamic response during storm events at reflective beaches with XBeach is difficult. The findings of Van Rooijen (2011) and especially Vousdoukas *et al* (2011), regarding difficulties with applying XBeach to reflective beaches, is discouraging in view of the many reflective beaches found along the South African coast. Corbella and Stretch (2012b) applied the XBeach model to predict erosion along the Durban coast. After significant calibration of the model, they found that the simulated erosion volumes were between 1% and 57% of the measurements. Corbella and Stretch also mention that the long simulation times was a disadvantage of the XBeach model. (In retrospect it may also be relevant to note that XBeach was (originally) developed to cater for applications with hurricanes, barrier islands, overwash and breaching, which are uncharacteristic of the South African coast.) In view of the various unfavourable findings and practical difficulties with applying the model in the South African context, it was decided not to persue the XBeach model further.

Another of the more modern and thus possibly better models, which was also accessible, is that by Van Rijn (2008, 2009). According to Van Rijn, his model is focussed on the cross-shore modelling of erosion “using a process-based profile model (CROSMOR2007-model), which has been extended to include four dominant dune erosion processes”. The CROSMOR model has been extensively verified (Van Rijn, 2008). Based on the results of a detailed sensitivity study, Van Rijn concluded that “the two most influencial parameters are the storm surge level (above mean sea level) and the bed material diameter, while the wave period also has a marked influence”. According to Van Rijn (2008, 2009), his model “is most valid for erosion under major storms, but also yields realistic results for minor storm events based on a comparison with measured data from USA beaches”. Van Rooijen (2011), concluded that the Van Rijn transport model (which is also utilized in the CROSMOR model) is well able to predict erosive swash conditions. In a recent study by Li *et al* (2014) on the estimation of coastal dune erosion and recession due to storm events, they also mention the "computationally expensive nature" of XBeach and opted for the "simpler and faster" model by Van Rijn. Based on its accessability, free use, acceptable theoretical approach, and its extensive testing, Van Rijn's model therefore also apprears to be a suitable model to apply in investigating coastal erosion. Accordingly, Van Rijn's model is applied in this research by comparing the results from new methods (discussed in the following sections) to those of Van Rijn's model. This application of Van Rijn's model to South African study areas, is discussed in Section 6.2.4. Together with extensive comparisons to field measurements, this serves to assess the robustness and applicability of the new methods for a variety of environments that are representative of the South African coast.

Approach for developing new methods to predict shoreline erosion

Most of the aforementioned models are time-dependent two-dimensional models of beach morphology change/evolution, although some are incorporated into quasi- or full three-dimensional models. Thus, most of these models can theoretically predict in relative detail how the beach profile will change over time (mostly in the short-term) as the main drivers change (e.g. the wave conditions), which can indeed be used to derive extreme event predictions of the shoreline erosion for incorporation into the determination of erosion setbacks. Regarding the determination of setback lines, the focus on cross-shore sediment transport (and other) processes and profile dynamics is in terms of predicting erosion setback distances. So, for setback lines, only the total horizontal erosion distance is needed, while the step-by-step changes in the beach profile shape, as provided by two-dimensional models, is complementary.

More complex situations, for example, where both cross- and longshore hydrodynamic processes drive shoreline behavior, or where sediment transport patterns and beach morphology are affected by human interference, often require more sophisticated two- or three-dimensional hydrodynamic, sediment transport and morphologic modelling (such as the Delft3D suite of models; Deltares, 2011a, 2011b or XBeach; Roelvink *et al*, 2009). Such two- or three-dimensional modelling is mainly suitable for detailed investigations of relatively small study areas where simulations are limited to relatively short periods (usually not more than a few years). Although some "acceleration" schemes (e.g. schematizations and/or increased morphologic time-step schemes) have been developed that may in specific circumstances be applied to enable longer term simulations (e.g. De Vriend *et al*, 1993; Hanson *et al*, 2003; Ranashinghe *et al*, 2011), the two- or three- dimensional modelling is mostly unsuitable for simulation of large study areas or predictions employing long time series of input data.

The existing comprehensive (more process based) cross-shore sediment transport or morphology models are typically "data-hungry", require significant calibration and are largely suited to detailed studies of small areas. As a general rule, it is agreed that numerical models should be verified and calibrated, and model results (from cross-shore morphological models) must be verified against recorded erosion, to allow calibration of model parameters (e.g. Schoonees and Theron, 1995). However, this is problematic, in that recorded erosion data is only available for very few locations in South Africa; thus this calibration and verification is generally not viable. This is not only true for South Africa, for example, Li *et al* (2014) state that, "ideally, the model should be calibrated by historical measurements of coastal profiles before and after a storm, however, these measurements are not available at the study site, a common situation at most locations around the world". The reliability of the cross-shore modelling is also dependent on the accuracy of the wave modelling, which requires detailed inshore bathymetry to enable accuracy. In the South African context, such detailed inshore bathymetry data is almost invariably only available at some ports and harbours and is prohibitively expensive to acquire for larger stretches of coastline (including many urban and important rural areas). It should, therefore, perhaps come as no surprise, that only one of the at least seven setback studies conducted in South Africa since 2010, appear to have included any form of cross-shore morphological modelling (the five studies that were available for review are discussed in Section 2.3.2). The one study that did include such modelling (the DEAD & P, 2010 study, Section 2.3.2), only had three very limited study areas with a total shoreline length in the order of 3 km. In addition, these were areas that had been studied extensively before and had a wealth of data available (that would only be matched in small areas at very few other South African sites).

A few authors (such as Callaghan *et al*, 2008), have combined statistical simulations with time-dependent beach profile modelling (e.g. Kriebel and Dean's model; Kriebel, 1995). Similarly, Ranashingle, *et al* (2011) used a statistical wave model along with a dune erosion model (Larson *et al*, 2004) to estimate dune erosion at Narrabeen Beach over the long-term. Callaghan *et al* showed relatively good reproduction of extreme beach erosion and incorporated important other aspects not usually included, such as joint probabilities between event duration, spacing and grouping, tidal anomalies and others. However their intensive method includes an elaborate 8-step process (one of which involves an iterative 7 step-process of its own), and also required lots of input data as well as calibration.

The afore-mentioned difficulties clearly indicate that there are still gaps in our knowledge and abilities regarding quantification of cross-shore sediment transport and morphologic processes, and especially in our ability to effectively apply such methods over large study areas. In view of the difficulties associated with applying conventional (2D or 3D) modelling and the accompanying need for extensive (and expensive) data collection and calibration, an alternative approach may be more suitable. Thus, studies were directed towards new methods requiring less input data to quantify these physical responses, which could potentially also be suitable to larger scale approaches. One such approach has been put forward by Bosom and Jimenez (2011), who related the storm eroded volume to Dean's simple predictor (Dean, 1973), storm duration and beach slope. (Dean's predictor is based on the difference between the dimensionless fall velocity parameter ($D = H/w_f$) and its value at equilibrium (Dean, 1973).) H is the wave height and w_f is the sediment fall velocity.) While Bosom and Jimenez agree that this is a simplification of the actual profile response, they state that the objective is not to attempt to reproduce the full beach profile response to the storm waves, but to derive a good estimate of the expected erosion.

In similar vein, two alternative approaches, a statistical and a parametric approach are proposed in the Sections 6.2.2 and 6.2.3, towards addressing the needs expressed in the foregoing. In developing these methods, essentially a down-scaling approach is followed, utilizing semi-empirical relationships. This is a suitable approach because of the typically large spatial scales (from tens of kilometres to a few hundered kilometres) and long temporal scales (from a few decades to more than 100 years) on which the methods need to be applied to. The methods also needed to be suitable for a wave-dominated coast (as South Africa is) considering processes relevant to coastal erosion (waves being the dominant factor in coastline response in this case) with the purpose of predicting the large-scale behavior (erosion) of the shoreline (in a sense similar to other behavior models such as Unibest-CL, Lipack, Astima and Estmorph (Bosboom and Stive, 2014)). The data poor South African situation necessitates input reduction, which therefore requires process reduction in applicable methods (i.e. model complexity generally has to be reduced for lack of comprehensive input data required to simulate all detail processes). Finally, the application in South Africa regarding promulgated time-frames for setting and affordability of determining setback lines, demands efficiency (i.e. large scale applications with modest computational and manpower requirements). Thus, the approach followed was to develop new methods in accordance with these requirements. Although a prerequisite of the statistical model is both process knowledge and data knowledge, this model relies on understanding / predicting the shoreline behavior based on measured data (as discussed in detail in Section 6.2.2).

The basis of the parametric approach (as discussed in detail in Section 6.2.3) is that it should be able to describe the gross cross-shore processes and behaviour of the shoreline based on simplified parameterised functional relationships which reflect the morphologic phenomena on a larger scale (as also stipulated by Van Rijn, 1998). (According to Stive and Walstra, 1998, parametric models can also be considered as reduced process-based models, where the dominant processes are modelled by means of parameterization.) These approaches are generally not suitable or intended for detailed designs. To assess the veracity of the new methods they are compared to the more complex Van Rijn model (2008) and especially to field measurements, as discussed in the following sections and Section 6.2.4.

6.2.2 Statistical analyses of short-term shoreline variability and erosion prediction

Background

<u>Introduction</u>

Although changing continuously, shorelines may be in a state of long-term dynamic equilibrium in which the average configuration does not change over time. Successive beach surveys provide a good indication of the “stability” of a shoreline. Shoreline variation is usually quantified by measuring the horizontal distance from a fixed reference point on land (typically a survey station) to a selected contour at the time that a specific survey was done. This distance is then compared to other such distances determined from surveys undertaken at different dates. Thus a database of the shoreline locations and variations is created, which can then be analyzed statistically to determine parameters such as extreme variations, standard deviations, etc. In Figure 6.2, examples are shown of graphs of such recorded distances, showing the variation of these distances over time. If the distance generally increases over time (i.e. a positive slope of such distances), this indicates accretion as the beach becomes wider (Profile B – the blue line, in Figure 6.2). Similarly, distances having a negative general slope (Profile C – the green line, in Figure 6.2), indicates erosion of the beach as the beach becomes narrower. If the distance varies around a mean value (having a horizontal trend on average), this indicates that the beach is dynamically stable (Profile A – the red line, in Figure 6.2), with no progressive eroding or accreting trend.

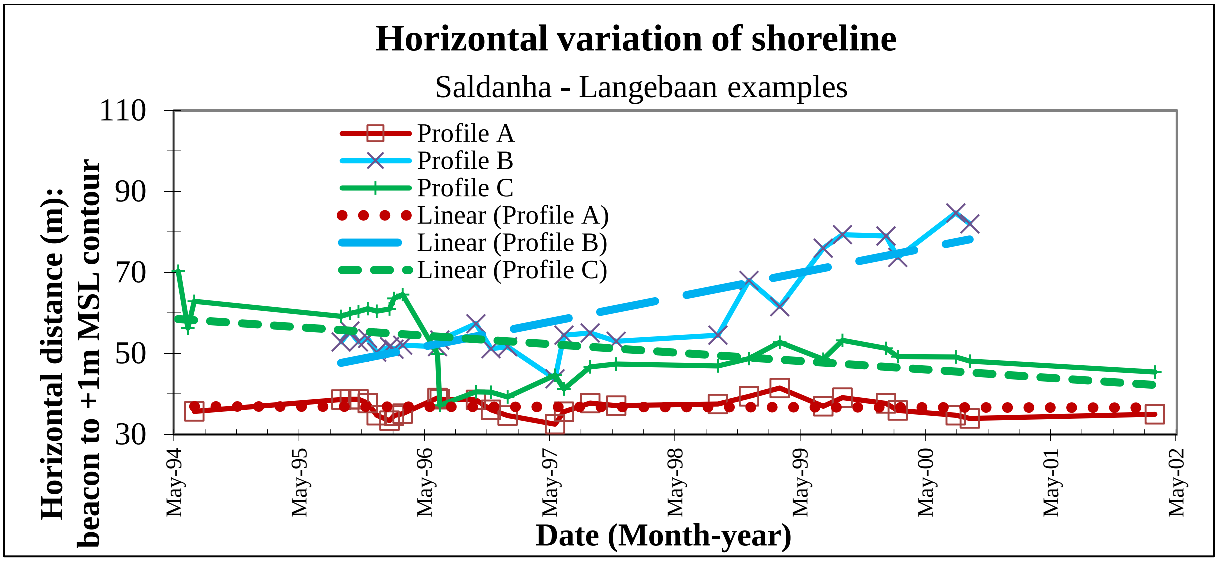


Figure 6.2: Example plots of horizontal shoreline location variations (distances) over time.

Premise

If the variation of a particular contour (for example, the shoreline, or say the +3 m CD contour) follows a known statistical distribution, this can be used to predict the maximum landward movement of this contour over a certain period (say 50 years). This implies the assumption that other factors (e.g. the metocean climate (winds, waves, etc.), tidal levels, sediment characteristics, etc.) will not change over the prediction period, which in most South African situations is expected to be true, with the exception of specific known or expected changes, such as for example SLR which can be accounted for separately. A further assumption is made that the beach material is homogeneous, which assumption would be true in many areas and is also assumed in most other methods of predicting coastal erosion. Therefore, the erosion setback related to short-term cross-shore shoreline variations (for example due to erosion/accretion resulting from sea storms and post storm recovery) can be determined if it can be shown that the variation follows a known statistical distribution and the parameters of this distribution have been quantified. In data poor environments, such as most of the South African coast, numerous beach surveys at one location over an extended period of time are usually not available to confirm the type of statistical distribution of the shoreline variation. However, three locations, namely Durban, Saldanha Bay and Richards Bay were identified (Figure 6.3), where ample good quality data was available (i.e. numerous surveys over a prolonged period), thus presenting an exceptionally good opportunity to investigate the statistical distributions of shoreline change in some South African locations.

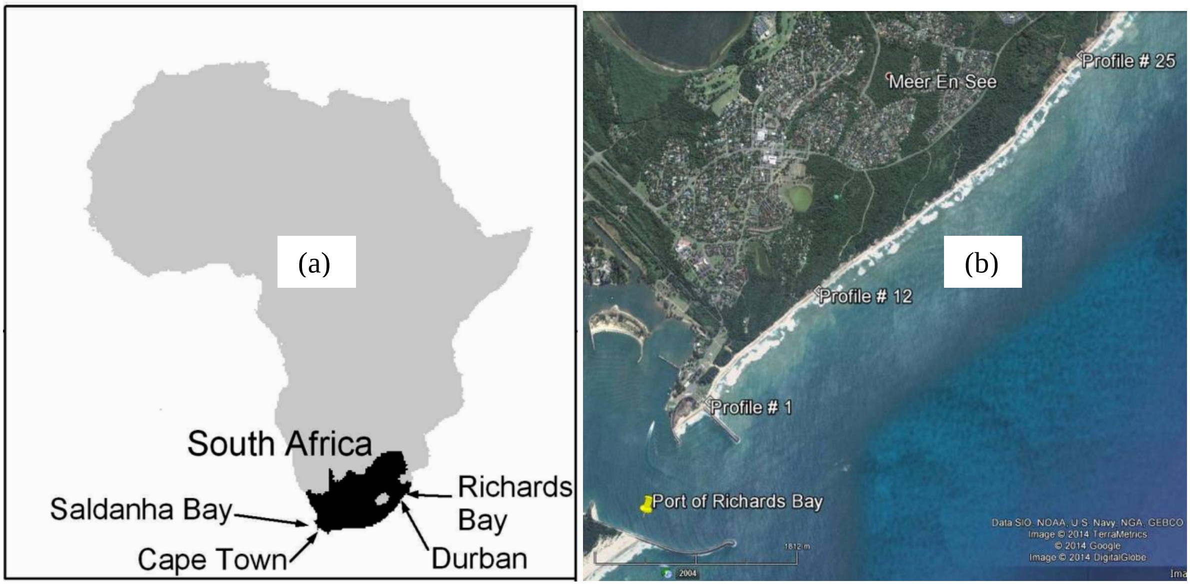


***Figure 6.3: (a) Coastal locations where sufficient beach survey data was available for analyses; (b) Profile locations at Richards Bay** (Google Earth)*

Data analysis

<u>Durban</u>

At Durban on the South African east coast (Figure 6.3), beach surveys of a variety of beaches have been conducted since 1973, creating a very large database. As of 2005, the eThekwini Municipality's extended coastal monitoring programme includes surveyed profiles of the beaches located to the south, within and north of the Durban Bight. For this study, all the profiles were selected that had records going back to at least 1989 (i.e. data record of 23 years or more). These profiles included 8 fully exposed open coast locations along the Durban Bluff ("Brighton": B6 – B13), 8 partially exposed open coast locations along the Virginia - outer Durban Bight ("Durban North": DN6 – DN13), and about 30 locations within the inner Durban Bight ranging from partially exposed near the Mgeni River Mouth to relatively sheltered locations near Vetch's Bight (numbered A to G and 1 to 23, in order of increasing wave shelter). The locations of the "Brighton" (B6 – B13), "Durban North" (DN6 – DN 13), and the inner Durban Bight profiles (A to G and 1 to 23) are indicated if Figures 6.4 and 6.5 respectively.

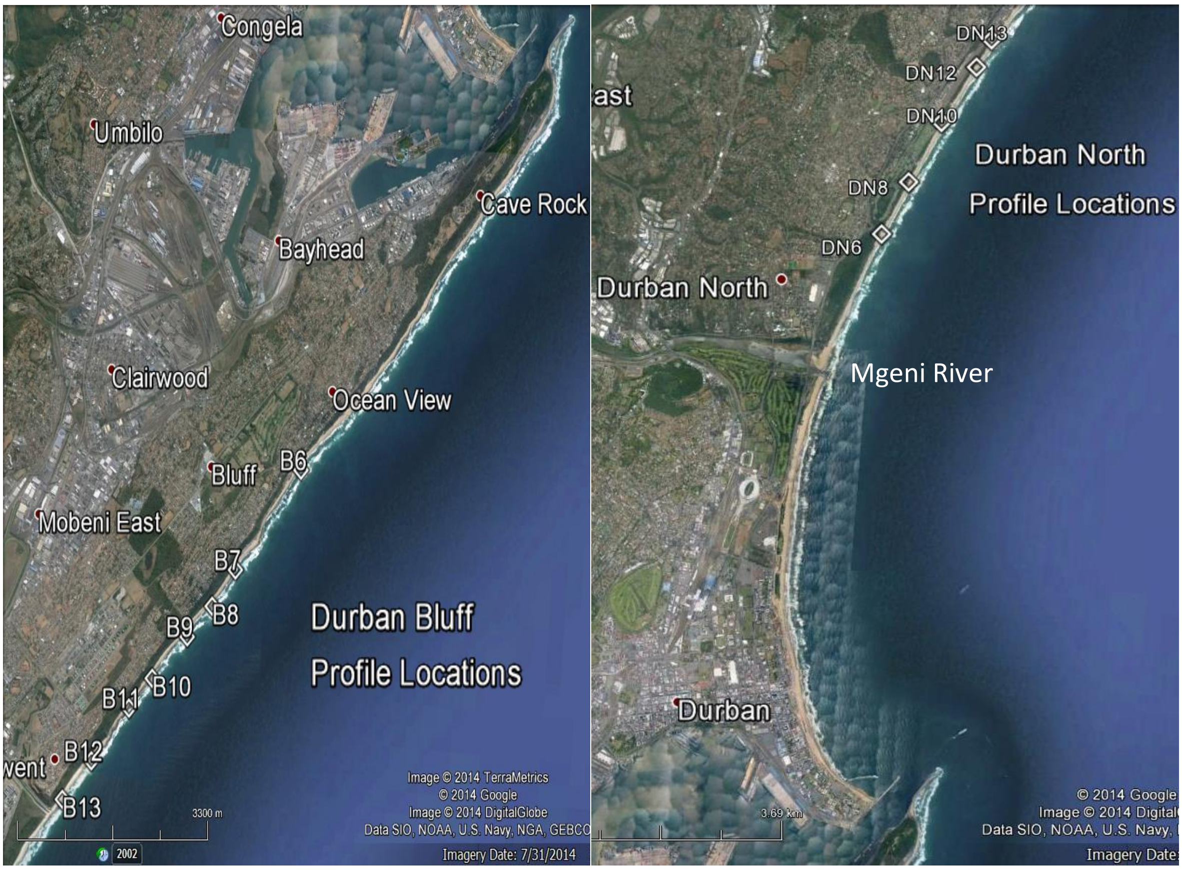


Figure 6.4: Locations of surveyed profiles along the Durban Bluff and Durban North. *(Google Earth)*

The Brighton data (B6 – B13) starts in 1989 and contains about 111 surveyed profiles per location, the Durban North data (DN6 – DN 13) begins in 1992 and contains about 99 surveyed profiles per location, and the inner Durban Bight profiles (A to G and 1 to 23) goes back to 1973 and contains at least 266 surveyed profiles per location.

Some of the Bight profiles were however eliminated from the analyses, as these are not "natural" beaches in the sense that their morphology and responses to natural marine drivers (e.g. erosion/accretion resulting wave action) are largely affected by direct anthropogenic interventions. Specifically, Profiles 13 to 23 (Figure 6.5) is where the bulk of the sand from the harbour sand bypassing scheme is pumped to (Mather *et al* 2003), which directly affects their on/offshore movement. (In the southernmost portion of the Durban Bight, shoreline configuration is directly dependent on the sand nourishment (harbour bypassing) scheme, especially the rate and type of sediment supplied (e.g. Theron *et al*, 2008).) Similarly, Profiles 2 to 11 (Figure 6.5) are in the direct vicinity of the Durban piers (which function as groynes, Campbell *et al*, 1985), which directly affects their on/offshore movement (in addition to still some (but a diminishing) effect of the sand pumping scheme). Finally, it may be observed that fluvial sediment inputs into the Durban Bight from the Mgeni River (Figures 6.4 and 6.5) could potentially affect shoreline variations at the profiles located nearest to the river mouth. Profile A is located nearest to the Mgeni Mouth at about 270 m south thereof, and is therefore the most susceptible to potential effects related to fluvial sediment inputs. However, Theron *et al* (2008) have calculated that the five large dams constructed on the Mgeni River between 1965 and 1988 respectively trap from 87% to 100% of the sand load in the river, and that since 1988 very little sand still reaches the sea. In addition, the net longshore transport direction at the Mgeni Mouth is towards the north and away from Profile A. Therefore, over at least the past 27 years (since 1988) the Mgeni River has had an insignificant effect (if any) on the beach profile variations in the Durban Bight area to the south of the river mouth. Thus, the data from the 9 remaining profiles (A to G and 1) was suitable for analyses of the inner Bight, in all therefore 25 suitable profiles from the 3 areas (8 from Brighton plus 8 from Durban North plus 9 from Durban Bight).

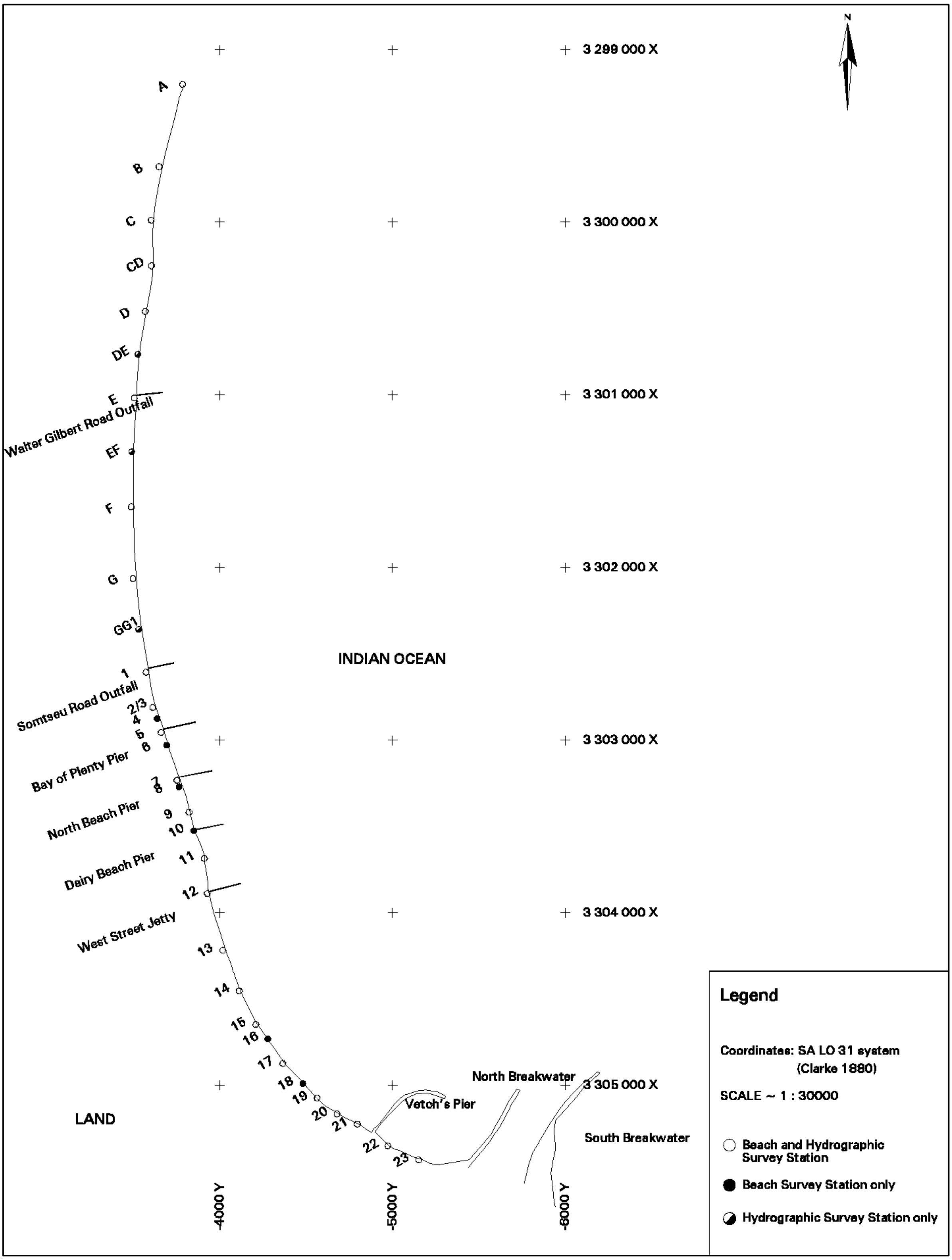


Figure 6.5: Locations of surveyed profiles (A to G, 1 to 23) along the inner Durban Bight. *(Theron et al 2010b)*

The shoreline variations were thus determined at the 25 profiles (beach cross-sections) along the 3 Durban study areas by analyzing, in each case, the horizontal distances from the survey station (located on the upper beach) as determined from each survey. An example of such analyses for Profile B10 along the Durban Bluff (Brighton) is shown in Table 6.1.

Table 6.1: Shoreline variation along the Durban Bluff (Brighton) at B10 from topographic surveys.

All distances are relative to the survey beacon (located on the upper beach) in meters calculated from beach surveys conducted within the period from 1989 to 2012.			
Elevation (m to CD)	Median distance (m) (of all the recorded distances measured between the selected contour line and the survey beacon.)	Standard deviation (m)	Number of values
3.0	22.1	6.3	106
2.0	30.6	6.5	106
1.0	47.6	6.7	95

The shoreline variations at Durban North Profiles N7 to N13 (based on all the beach surveys and calculated in the same manner as before) are shown in Table 6.2.

Table 6.2: Shoreline variation along Durban North (outer Bight) at Virginia from topographic surveys.

+3 m CD contour distances (m) relative to survey beacons			
Profile	Median distance	Average distance	Standard deviation
N6	78	65	15
N7	41	32	10
N8	42	41	7
N9	32	32	7
N10	45	45	8
N11	37	32	7
N12	29	31	7
N13	39	33	8

To investigate the type of statistical distribution of the shoreline variations, a normal distribution ("Gaussian" or bell shaped) was fitted to the data for the 3 Durban study areas. Examples of how well the normal distribution fits the data for each of the 3 areas are shown in Figures 6.6 to 6.8 (Profiles B7, DN8 and G, respectively), while examples of inadequate fits for each study area are shown in Figures 6.9 to 6.11 (Profiles B13, DN11 and B, respectively). Chi-square goodness-of-fit tests (Walpole and Myers, 1978; Kreyszig, 1970) were conducted on all 25 profiles to confirm whether the data met the criterion for normal distributions. It was found, at a 95 % confidence level, that the short-term shoreline variations at 19 of the 25 profiles are indeed normally distributed. In other words, the statistical distributions of the shoreline variation of the Durban beaches indicate that a normal distribution, at a 95 % confidence level, may be assumed for the short-term shoreline variations at most of the profiles (76%). The standard deviation on the more exposed, natural beaches around Durban (i.e. Brighton and Durban North) is 6 m to 15 m.

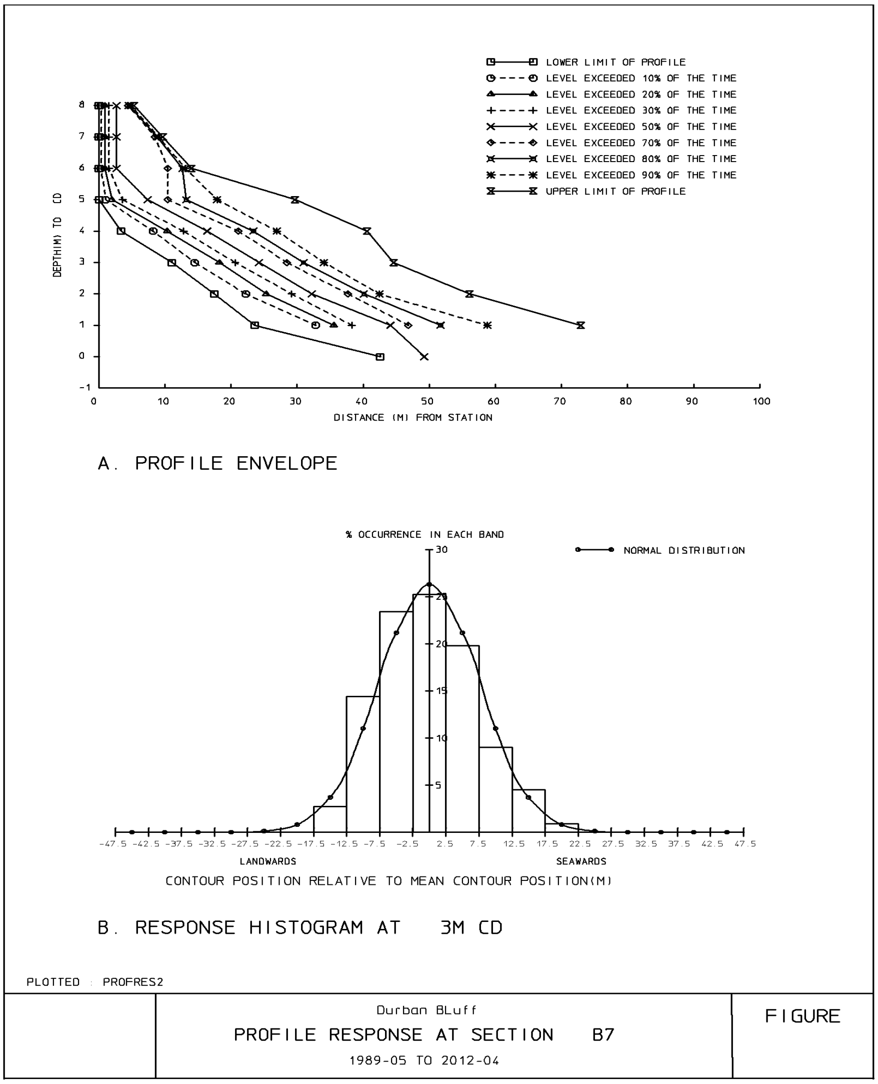


Figure 6.6: Profile envelopes and distribution of cross-shore variations at Profile B7 (Brighton).

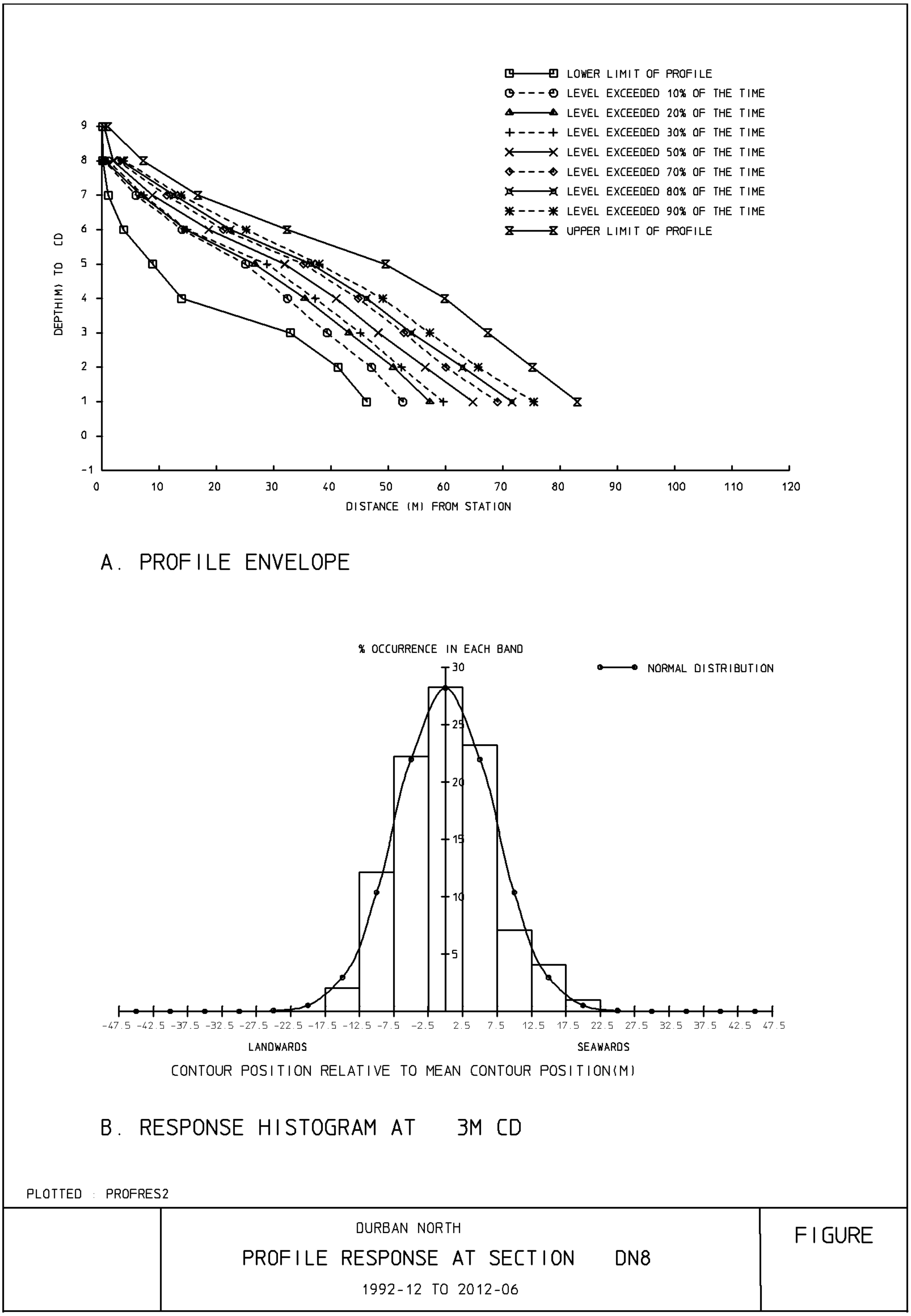


Figure 6.7: Profile envelopes and distribution of cross-shore variations at Profile DN8 (Durban North).

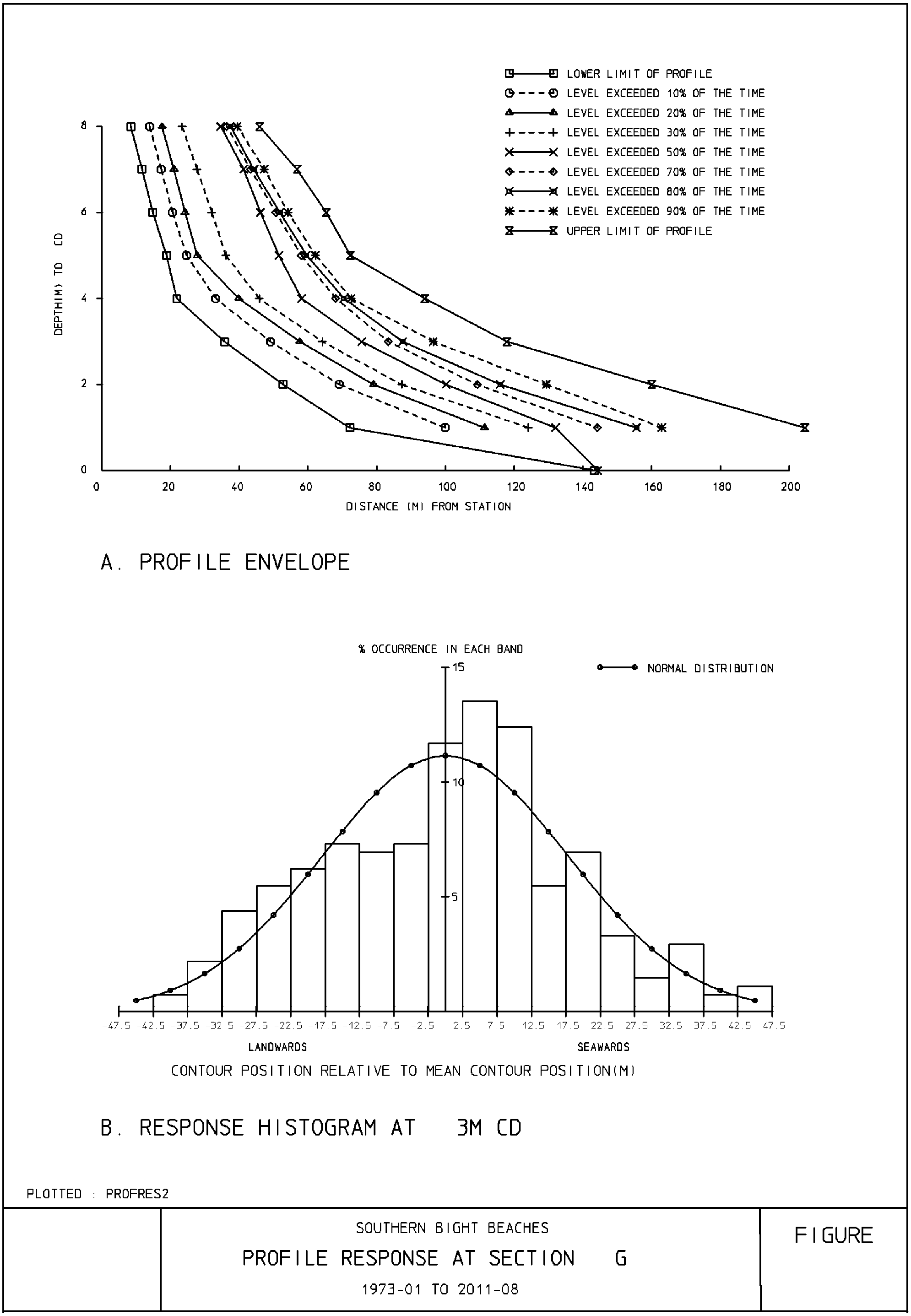


Figure 6.8: Profile envelopes and distribution of cross-shore variations at Profile G (Durban Bight).

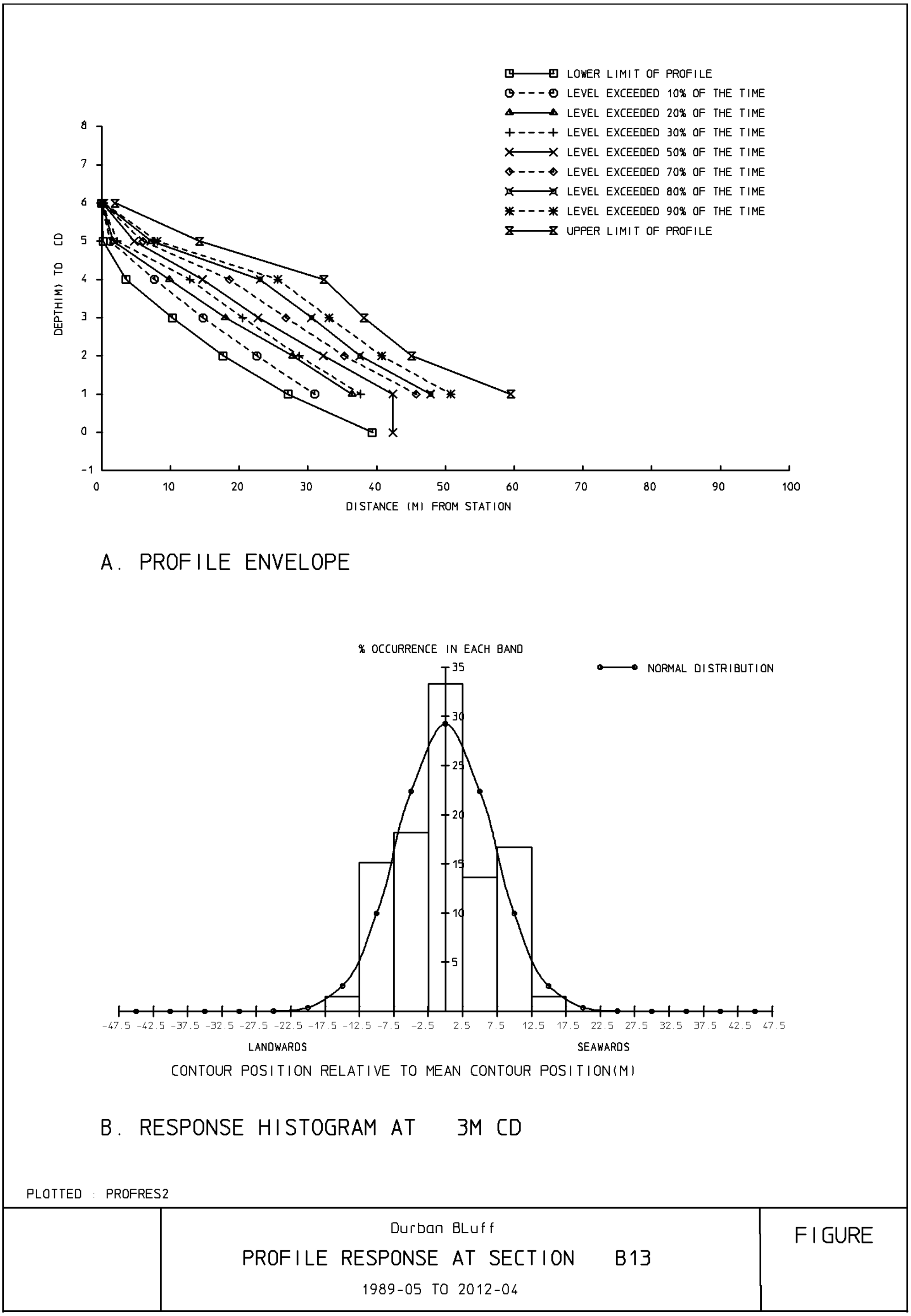


Figure 6.9: Profile envelopes and distribution of cross-shore variations at Profile B13 (Brighton).

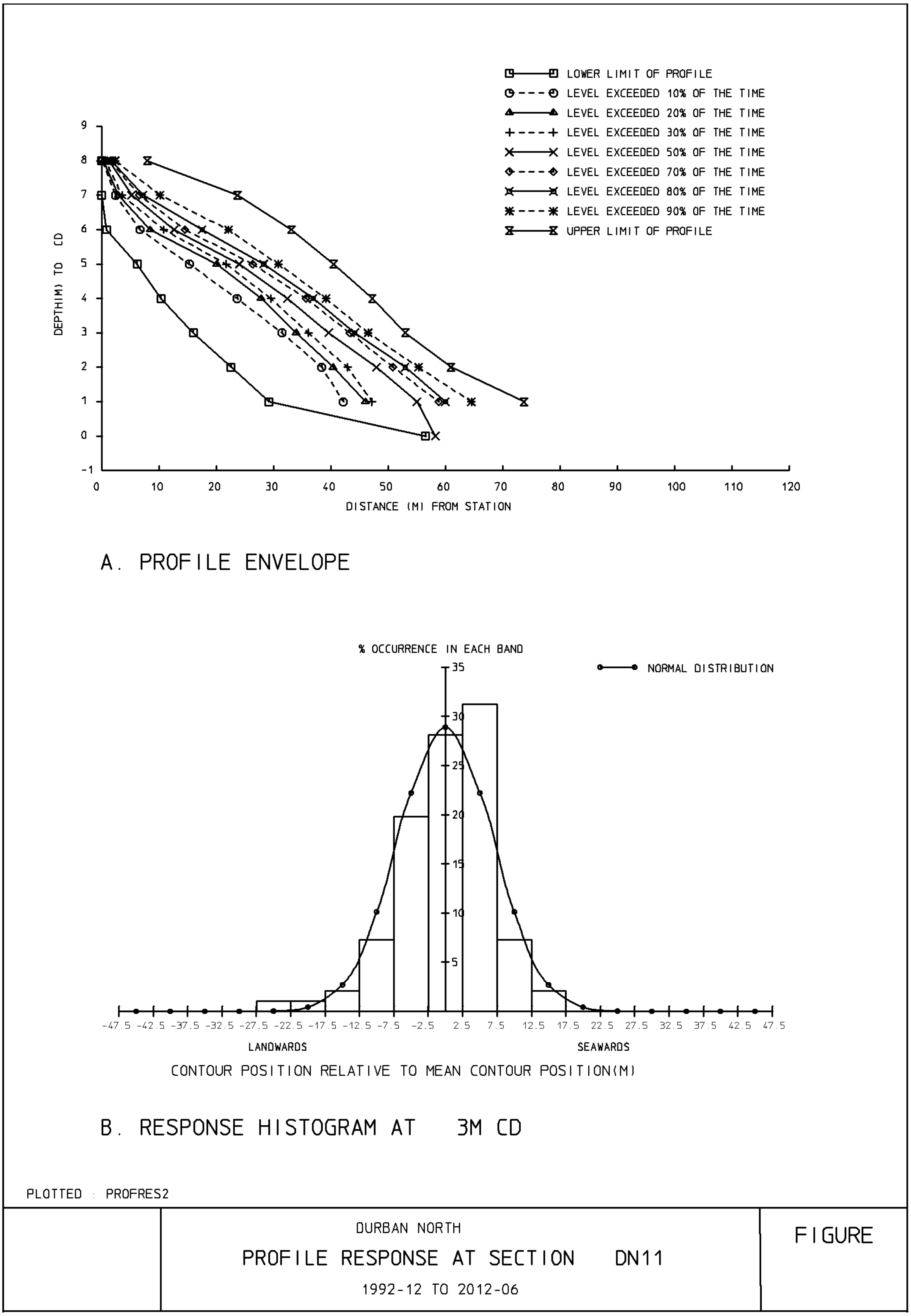


Figure 6.10: Profile envelopes and distribution of cross-shore variations at Profile DN11 (Durban North).

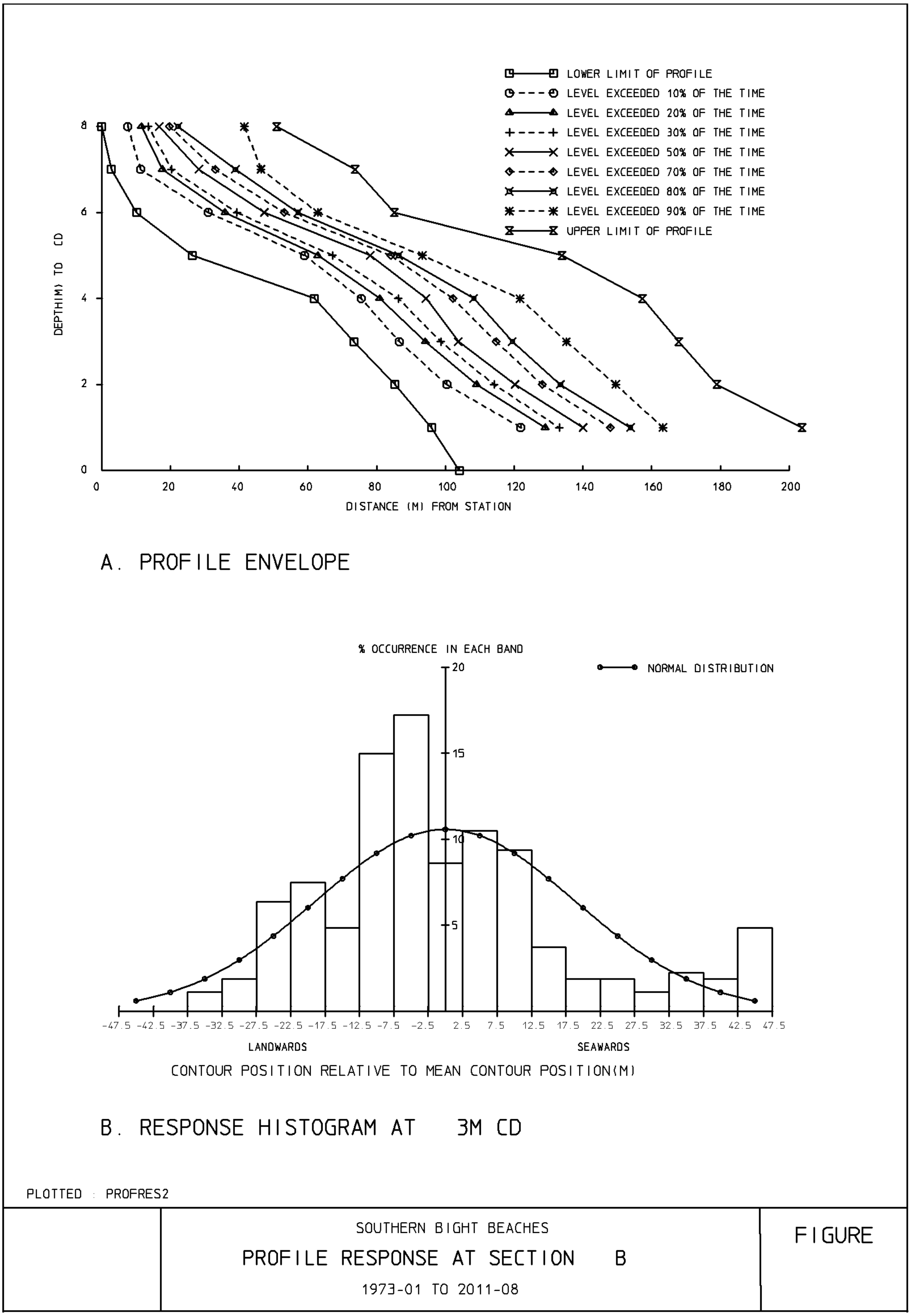


Figure 6.11: Profile envelopes and distribution of cross-shore variations at Profile B (Durban Bight).

Regarding the Brighton profiles, of which only B13 failed the chi-square test, it can be said that although it did not strictly meet the criteria at a 95 % confidence level, it did not fail the test by a large amount (sum of chi-squares = 13.7 versus acceptable sum = 9.5), and the normal distribution still fits the data to some degree (Figure 6.8). Similarly, of the three Durban North profiles that failed the chi-square test, DN6 and DN9 failed the test by relatively small amounts (sum of chi-squares = 21.6 and 13.3 versus acceptable sums = 18.3 and 7.8, respectively). Even the worst of these three in terms of the chi-square test (DN11) still seems to exhibit some degree of fit to the normal distribution (Figure 6.10).

The two profiles that did not adequately fit the normal distribution within the inner Bight area, were profiles B (Figure 6.11) and C. It was subsequently discovered that the survey base stations (beacons) had been moved in the cross-shore direction during the monitoring period by as much as 40 m to 50 m (A Mather, pers com). This would have a major effect on the statistics of the cross-shore variations, and is the most apparent reason for these profiles deviating from the relatively good fit to a normal distribution exhibited by the adjacent and very similar profiles.

Wiegel (1964) classified beaches as being either exposed, moderately protected, or protected from wave action. Based on this classification, and in view of the above results and discussions, it can be said that the shoreline variations are normally distributed on both the exposed (Brighton and Durban North) and moderately protected (inner Durban Bight) beaches near Durban, at the majority of beaches (at least 76%), but arguably at almost all of the beaches (≥ 19 out of 23, i.e. ≥ 83%).

Saldanha Bay Shorelines

Beach profiles were surveyed in Saldanha Bay from 1994 to 2002 (28 surveys). The nine years of survey data is sufficient to enable beach stability analysis and statistical determination of shoreline changes in the short term. (These surveys were conducted to determine, amongst others, whether harbour development has had any noticeable effect on the surrounding beaches in Saldanha Bay (CSIR, 2000)). Profiles or cross-sections were measured opposite five survey stations along the study area (called Profiles 1 to 5). The locations of these beach cross-sections are shown in Figure 6.12. As an example, a few beach profiles measured opposite Station 3 are shown in Figure 6.13. The shoreline variation was thus determined at the five cross-sections along the study area by measuring, in each case, the horizontal distance from the survey station to the +1 m to mean sea level (MSL) contour (which is near the spring high-water level for Saldanha Bay) as determined from each survey. The shoreline variations at Profiles 1 to 5 (based on all the beach surveys) are shown in Figure 6.14 and listed in Table 6.3.

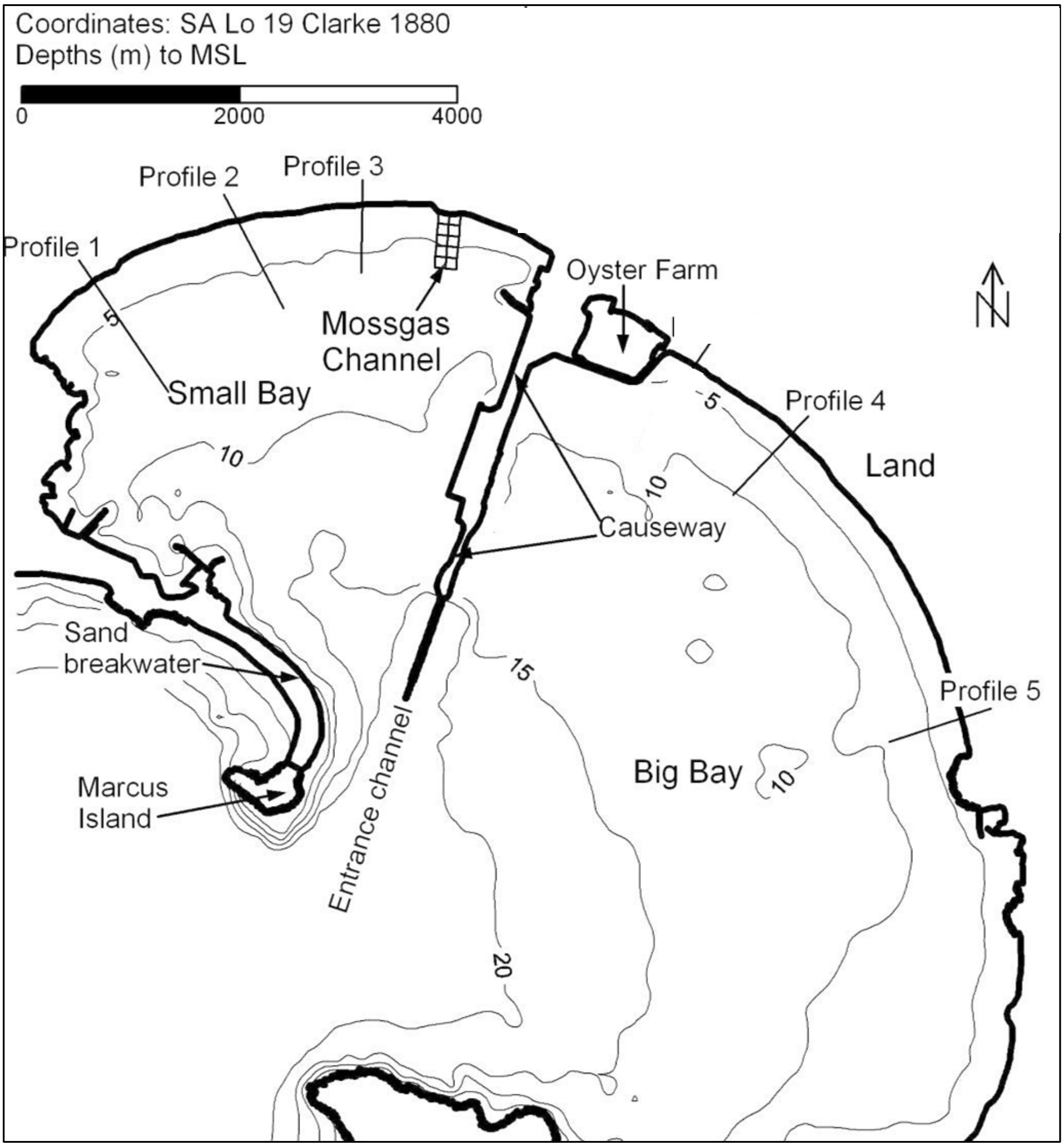


Figure 6.12: Locality map of beach profiles surveyed in Saldanha Bay

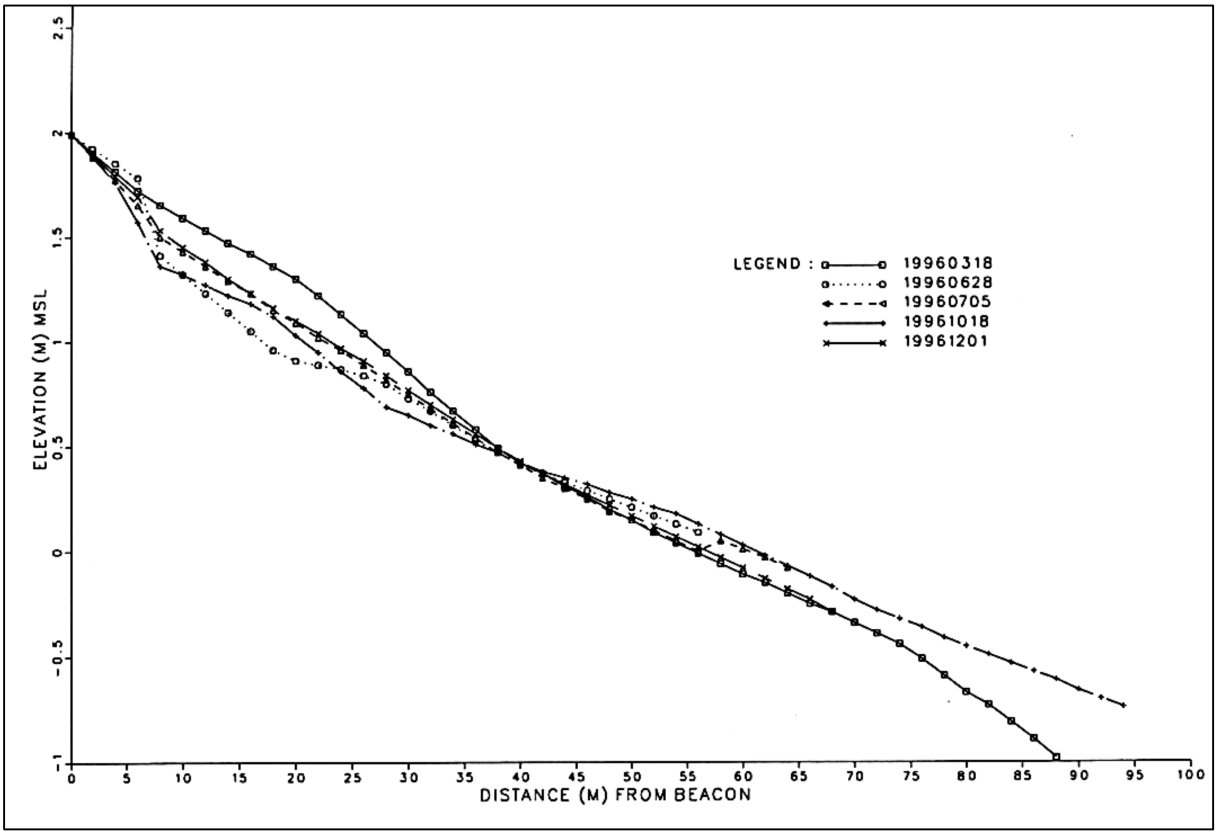


Figure 6.13: Beach profiles at Station 3 surveyed on the northern shore of Saldanha Bay

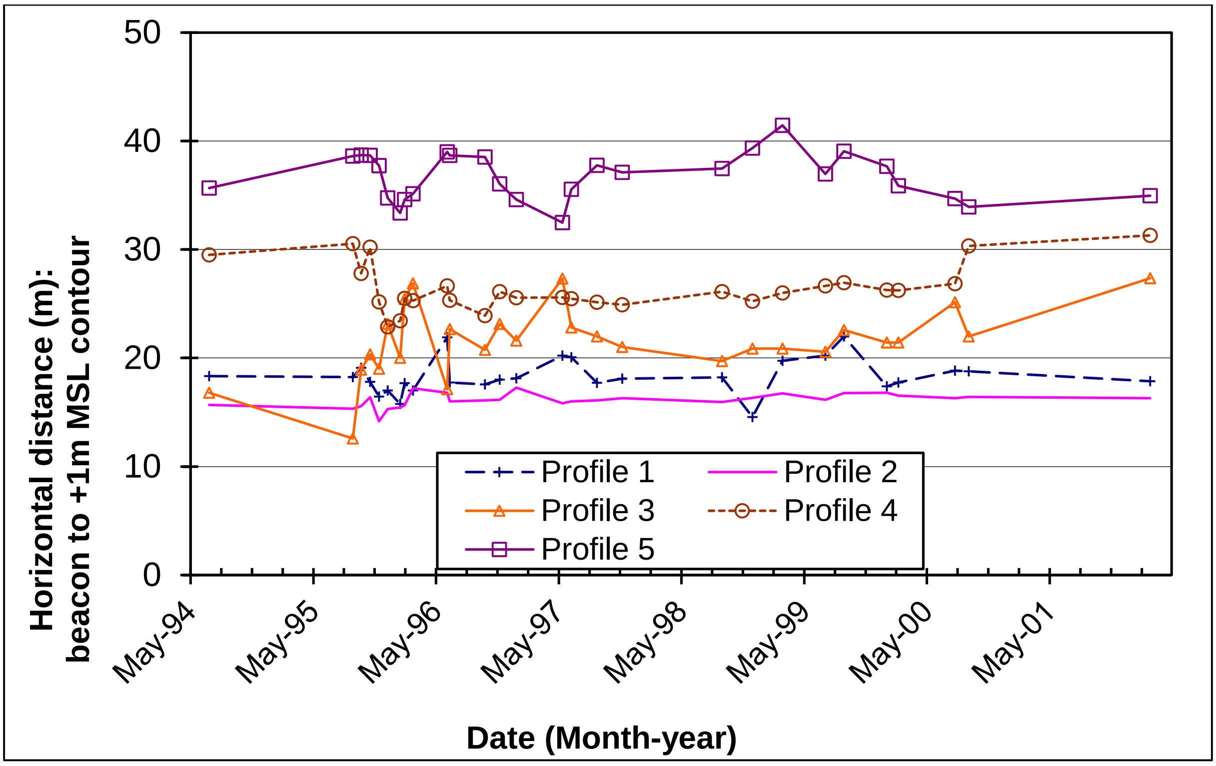


Figure 6.14: Shoreline variations at Stations 1-5 surveyed in Saldanha Bay

Table 6.3: Saldanha Bay Shoreline Variations *(distances between +1 m MSL contour and fixed survey beacons located on upper beach)*

Profile	Number of surveys	Minimum distance +1 m MSL to beacon (m)	Maximum distance +1 m MSL to beacon (m)	Maximum variation = maximum distance – minimum distance (m)	Median distance (m)	Standard deviation of the distances (m)
1	28	15	22	7	18	1.6
2	28	14	17	3	16	0.6
3	28	13	27	15	21	3.2
4	28	23	31	8	26	2.1
5	28	32	41	9	37	2.2

The beach profiles and profile envelopes for Profiles 1, 2 and 3, as well as the low standard deviations of the distances for these profiles in Table 6.3, indicate small profile variations. This is due to the sheltered location of these profiles within Small Bay (Figure 6.12). Similarly, Profiles 4, and 5 in Big Bay (Figure 6.14 and Table 6.3) indicate relatively small profile variations. These profile variations are somewhat larger than those in Small Bay, but still much smaller than those found along exposed beaches of South Africa. For example, the standard deviation on the exposed, natural beaches around Durban is 6 m to 15 m (see previous section). The statistical analyses of the shoreline variations presented in Table 6.3 provide further confirmation of these findings. Profiles 1 to 5 have small maximum variations (between 3 m and 15 m) and small standard deviations (up to only 3.2 m).

A normal distribution has also been fitted on the data for the Saldanha Bay study area. An example of how well the normal distribution fits the data for Profile 3 is shown in Figure 6.15. From this figure, it appears that the statistical distribution of the shoreline variation of this beach is also approximately a normal distribution. Both chi-square and Kolmogorov-Smirnov goodness-of-fit tests (Kreyszig, 1970) were conducted on all 5 profiles to confirm whether the data met the criterion for normal distributions. It was found, at a 95 % confidence level, that the short-term shoreline variations at Profiles 1 to 5 (dynamically stable beaches) are indeed normally distributed. In other words, the statistical distributions of the shoreline variation of the Saldanha Bay beaches in dynamic equilibrium indicate that a normal distribution, at a 95 % confidence level, may be assumed for the short-term shoreline variations at Profiles 1 to 5. Based on Wiegel's (1964) classification, it can be said that the shoreline variations are normally distributed on both the protected (Small Bay) and moderately protected (Big Bay) beaches at Saldanha Bay.

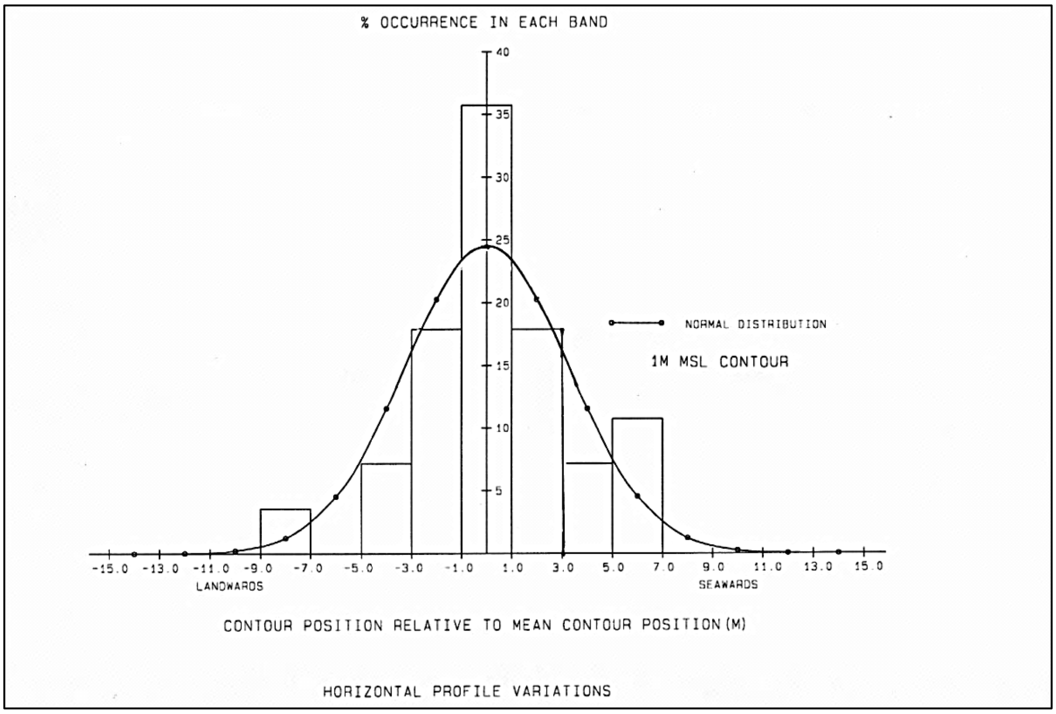


Figure 6.15: ***Normal distribution of shoreline variation at Station 3 located on the northern shore of Saldanha Bay***

Richards Bay

Measurements were made along the Richards Bay shoreline (Figure 6.3b) at 25 locations on 54 occasions between January 1989 and February 2004. The beach surveys were conducted on average about every 3.5 months, which is therefore well representative of all seasons of the year (storm wave occurrence in South Africa generally has a significant seasonal signature). Previous analyses of the survey data have indicated a general long-term erosional trend in the order of 1 m to 2 m per year (largely ascribed to insufficient sand bypassing of the port; CSIR, 2005), as is also evident in Figure 6.16 for example.

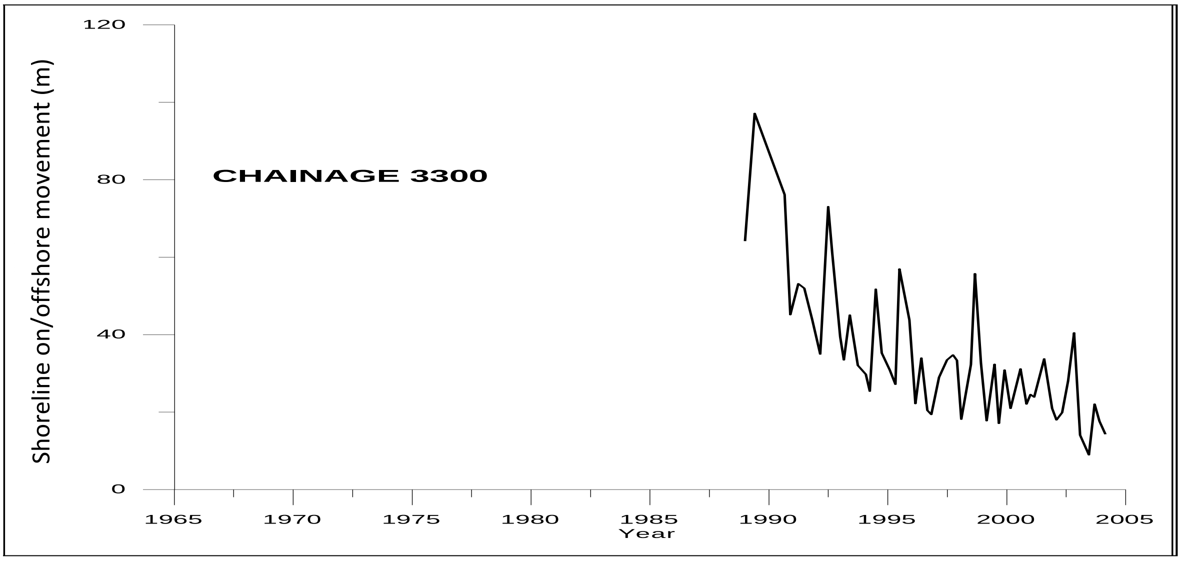


Figure 6.16: Shoreline (+1 m CD) on/offshore movement at 3300m N of the North Breakwater at Richards Bay *(CSIR, 2005)*

To assess short-term cross-shore changes, shoreline variations were statistically analysed (relative to the long-term trend). For the 25 locations, the standard deviations of these cross-shore variations were extracted. As before, the chi-square goodness-of-fit test (Walpole and Myers, 1978) was also conducted on all 25 data sets, to assess whether the variations (about the long-term trend) were normally distributed.

The results of the statistical analyses of the cross-shore shoreline variations are shown in Table 6.4, which also indicates whether the short-term variations met the criterion for a normal distribution. Up to a distance of 2.5 km north of the northern breakwater (Profiles 1 to 18), all 18 profiles exhibit a normal distribution of short-term variations, while the variations of the 7 profiles located further north (Profiles 19 to 25) do not conform to the chi-square goodness-of-fit test (at 95% confidence level) for a normal distribution. Thus, it is found that the short-term shoreline variation of the Richards Bay beaches is normally distributed at 18 out of 25 locations. At the few (28%) locations where it is not normally distributed, this is due to the "total" loss of the former beaches through extreme progressive erosion into the dune "under-layers" (exposing more resistant mudstone and cohesive sediment layers that inhibit rapid erosion), as illustrated in Figure 6.17. This extreme "sand stripped" shoreline has characteristics unlike those found at virtually all other "natural" sandy beaches in South Africa.

Table 6.4: Short-term shoreline variations north of Richards Bay harbour

Profile number	Distance alongshore from North breakwater (m)	**Standard deviation of cross-shore distances (m** between fixed survey beacon and +1 m CD contour,	**Meets criterion for a normal distribution**
1	0	15	Yes
2	30	14	Yes
3	144	13	Yes
4	258	15	Yes
5	372	23	Yes
6	467	16	Yes
7	562	13	Yes
8	684	11	Yes
9	805	10	Yes
10	1001	9	Yes
11	1152	8	Yes
12	1430	9	Yes
13	1592	7	Yes
14	1763	8	Yes
15	2026	7	Yes
16	2185	6	Yes
17	2387	6	Yes
18	2556	6	Yes
19	2736	8	No
20	2944	8	No
21	3266	11	No
22	3641	8	No
23	3909	9	No
24	4192	7	No
25	4554	8	No

Figure 6.17: Loss of former beach north of Richards Bay through extreme erosion into dune "under-layers" exposing mudstone and cohesive sediment layers.

General discussion regarding statistical analyses of South African shoreline variations

Based on the foregoing analyses, it is concluded that the short-term shoreline variation of protected, moderately protected, and exposed, natural beaches in South Africa is mostly (>72%) normally distributed. Although this finding has, strictly speaking, only been proven for the areas tested at Durban, Saldanha Bay and Richards Bay, it is reasonable to assume that the behaviour of other similar South African beaches is alike. This assumption is given further credence by the fact that the numerous individual locations tested (53) are situated in different coastal regions, are subject to a wide range of metocean drivers (e.g. incident wind and wave climate, wave exposure/shelter, etc.), and exhibit a great variety of characteristics (e.g. beach slopes, sand grain sizes, morphology). It is nevertheless recommended that this behaviour be investigated further to confirm that short-term shoreline variations on most other South African beaches are also normally distributed. It should be pointed out that the conclusion that short-term shoreline variations along the South African coast are normally distributed, does not in any way imply that all (or even many of) the beaches are dynamically stable. Some are indeed progressively eroding while a few others are accreting in the long-term. However, besides or on top of these (background) long-term trends, it is found that the natural short-term erosion/accretion variability displayed (mainly due to sea storms and post storm recovery) is mostly normally distributed.

A proposed statistical model to predict short-term cross-shore erosion

Based on the conclusion that the variation of the shoreline follows the normal statistical distribution, this can now be used to predict the maximum landward movement over a selected period (say 50 years). According to the continuous normal probability distribution (e.g. Walpole and Myers, 1978) it can be stated that:

$$\mathbf{Z} = (\mathbf{X} - \boldsymbol{\mu}) / \boldsymbol{\sigma}$$

Where the probability of **Z** is given by the area under the normal curve,

X is the value of the variable, in this case the shoreline variation,

μ is the mean, in this case of the shoreline variations, and

σ is the standard deviation, in this case of the shoreline variations.

This equation can be transposed to yield X, or the predicted shoreline variation as follows:

$$\mathbf{X} = \mathbf{Z} \cdot \boldsymbol{\sigma} + \boldsymbol{\mu} \qquad \textit{(Equation 6.1)}$$

In other words, the predicted shoreline variation (X), is equal to the Z value given by the chosen probability, multiplied by the standard deviation (σ), plus the mean (μ), both of the shoreline variations. The Z values can be read off standard statistical tables, such as Table IV published in Walpole and Myers (1978; p513); for example, for a chosen P of 0.95 (i.e. 95%), the Z values is 1.645. Assuming for the purposes of this example that the standard deviation was found to be 10 m, this would mean that the variation (offset from the mean) would not exceed 16.45 m (= 10 m x 1.645) for 95% of the time. This offset from the mean, taken in a *landward* direction, therefore constitutes the predicted erosion. Thus, in this example, the *erosion* would not exceed 16.45 m for 95% of the time. In other words, the predicted shoreline erosion (landward variation) for a chosen exceedance probability is given by Equation 6.1.

To test the veracity of this method of predicting erosion, data sets were selected from the Durban Bight and Durban North long-term profile data base as described before in Section 6.2.2. The data sets were selected on the basis of having at least 100 data points each of recorded shoreline location distances from their respective base stations, to ensure that the predicted erosion could be compared with accurate erosion data. Percentile values of horizontal distances (at +3 m CD) were then determined from these data, namely the 50%, 75%, 90%, 95%, 99%, 99.3% and 99.6% values. As an example, these distances for Profile B6 are indicated in Table 6.5 (the green row). Thus, for example for B6, the 50- and 99-percentile values (distances to survey base station) were 34.7 m and 14.6 m respectively. The erosion distances for each of these values were then calculated relative to the 50% value. For example for B6, the 50- and 99-percentile values (distances relative to 50% value of 34.7m) were 34.7-34.7 = 0 m and 34.7-14.6 = 20.1 m respectively, as indicated in Table 6.5 (by the yellow row). These data as determined for each of the selected profiles, as well as the standard deviation values, are listed in Table 6.5.

Table 6.5: Shoreline variations (percentiles) recorded along the Durban Bluff and Bight

Station name			***Percentile value of horizontal distance to the beacon (m) for percentile:***						
			50	***75***	***90***	***95***	***99***		
B6: Durban Bluff			*34.7*	*30.0*	*26.7*	*24.5*	*14.6*		
Station name	**Number of values**	**Standard deviation (m)**	**Erosion distance from 50 percentile value (m) to percentile:**						
			50	**75**	**90**	**95**	**99**	**99.3**	**99.6**
Durban Bluff									
B6	107	7.6	0.0	4.7	7.9	10.2	20.1		
B7	107	7.4	0.0	4.6	9.2	10.8	12.4		
B8	107	7.8	0.0	5.3	10.0	14.3	17.1		
B9	107	7.1	0.0	4.7	10.9	12.7	17.9		
B10	106	6.3	0.0	3.2	6.0	8.6	14.9		
Durban Bight									
A	266	21.0	0.0	11.2	21.5	31.1	42.4		
D	266	13.0	0.0	8.9	13.1	15.4	24.3		
E	266	12.4	0.0	7.3	12.9	15.3	23.3		
F	266	14.2	0.0	9.4	14.0	17.9	26.4		
1	310	14.3	0.0	9.1	15.0	20.3	26.5	31.4	37.3

The Z values for the continuous normal probability distribution (e.g. from Table IV published in Walpole and Myers, 1978; p513) for the chosen percentiles, are listed in Table 6.6.

Table 6.6: Z values for the continuous normal probability distribution

Selected percentiles (%)						
50	75	90	95	99	99.3	99.6
Z values						
0	0.675	1.280	1.645	2.33	2.455	2.65

Based on Equation 6.1, the standard deviation values given in Table 6.5, and the Z values given in Table 6.6, the predicted erosion amounts can be calculated for each profile and percentile. The predicted shoreline erosions (at +3 m CD) for each of the selected profiles and percentiles are listed in Table 6.7.

Table 6.7: Predicted shoreline erosion along the Durban Bluff and Bight

Station name	**Percentile (%)**						
	50	**75**	**90**	**95**	**99**	**99.3**	**99.6**
Durban Bluff - Predicted erosion for each percentile (m)							
B6	0.0	5.1	9.7	12.4	17.6		
B7	0.0	5.0	9.4	12.1	17.2		
B8	0.0	5.3	10.0	12.9	18.2		
B9	0.0	4.8	9.1	11.6	16.5		
B10	0.0	4.3	8.1	10.4	14.8		
Durban Bight - Predicted erosion for each percentile (m)							
A	0.0	14.1	26.8	34.5	48.8		
D	0.0	8.8	16.7	21.4	30.4		
E	0.0	8.3	15.8	20.3	28.8		
F	0.0	9.6	18.2	23.4	33.2		
1	0.0	9.6	18.2	23.4	33.2	35.0	37.8

The erosion amounts (Table 6.7) predicted by the "Normal" model (Equation 6.1) are compared to the recorded erosion distances (Table 6.5) in Figure 6.18. Clearly the predicted erosions compare very well with the data (R^2=0.97), although the model does over-predict by a relatively small amount (about 14%) on average.

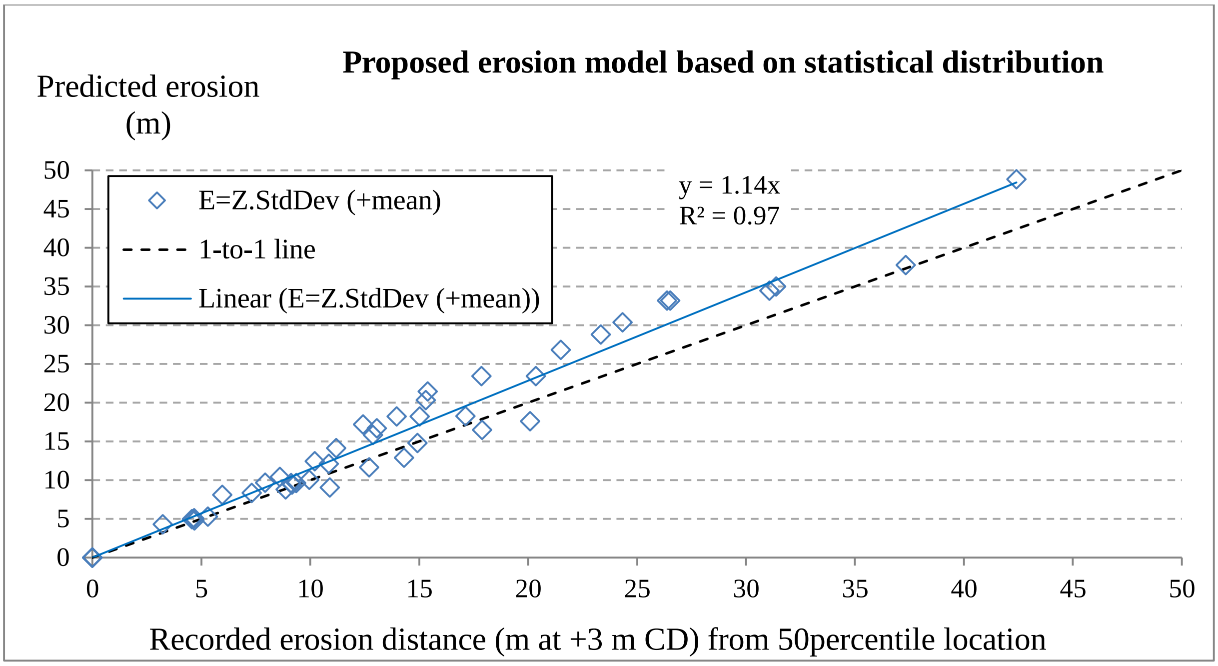


Figure 6.18: Comparison of recorded and predicted erosion distances based on the Normal model

It should be kept in mind that the two areas from which the profiles were selected are quite different in nature: the Durban Bluff area being an open fully exposed coast, while the other profiles are located in a moderately exposed location within the Durban Bight. In addition, the individual profiles have wide ranging characteristics with very different slopes, sediment grain sizes and morphologies. Thus, it seems that the model is relatively robust, providing satisfactory results under a relatively wide range of conditions.

Validation of Normal Model

To validate the satisfactory performance of the Normal model, additional tests were conducted on new sites located in areas which differ from the two areas were the model was applied above. Four profiles were selected from the exposed Virginia Beach area located to the north of Durban. The same procedure as described above was followed to analyze the recorded erosion data and to predict erosion with the Normal model. The only difference from before is that these recordings contained less data points, which meant that percentile values could only be determined up to 95% from the recordings (thus yielding a few less data points to compare with the modelled results). The erosion amounts predicted with the "Normal" model (Equation 6.1) are compared to the recorded erosion distances in Figure 6.19. Again it is clear that the predicted erosions compare very well with the data, with the model over-predicting by only a small amount (about 2%) on average.

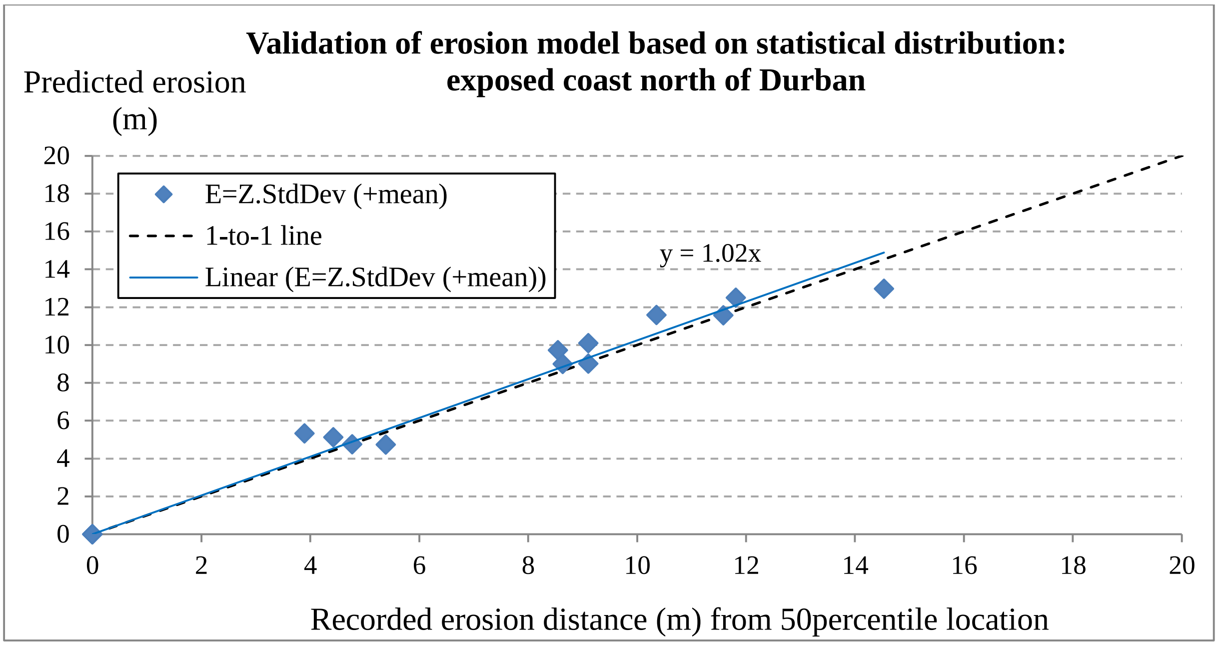


Figure 6.19: Recorded versus predicted erosion distances with the Normal model - Virginia

These satisfactory results give further credence to the preliminary conclusion that the Normal model is relatively robust, providing satisfactory results under a relatively wide range of conditions. To further confirm the general applicability of the model for use in South Africa, the model should be tested against additional data from other areas. Unfortunately, at this stage, there do not appear to be other South African sites where sufficient data has been recorded to allow robust comparisons to be made. Yet, based on the foregoing, the model is expected to be generally applicable for use in South Africa. A final verification of the model is conducted in Section 6.2.4, by applying the model to two new study areas and comparing the results with those of a comprehensive (i.e. process based approach) model, as well as to the Parametric model.

Cross-shore shoreline variations from aerial photography

In Section 6.2.2 a description is provided of how shoreline variation can be quantified by means of topographic surveys, if such data is available. However, for much of the South African coast, such data is not available. Fortunately, shoreline variation can also be quantified by means of vertical aerial photography (or suitable satellite imagery). The limitations of assessing shoreline variation and stability by means of aerial photograph analyses are the level of accuracy when establishing the position of the high water mark (accurate to within 10 m) and especially the availability of aerial photographs (for example, the number of photographs available covering the last 50 years). According to O'Connel (2003), the wet-line observed in aerial photography can be used to give a good approximation of the shoreline location (in this case at approximately the spring high tide wave runup line) along sandy areas at the time that an aerial photograph was taken. (Camfield and Morang, 1996, provide a further discussion of some of the issues around interpretation of shoreline changes from aerial photographs and maps.) In order to compare the aerial photographs quantitatively they also have to be converted to the same scale and any possible distortions have to be removed as far as possible.

While this discussion indicates that shoreline variation can indeed be quantified by means of vertical aerial photography (or suitable satellite imagery), this does not yet prove that such data can be used to predict short-term shoreline erosion. To investigate the utility of aerial photography data for this purpose, two areas were selected for which shoreline variations have already been analyzed based on topographic survey data. This was done so that the results from the aerial photograph analyses could be compared to accurate results based on a large quantity of good quality data. The areas selected for analyses of aerial photographs were Brighton and Virginia Beach. The analyses of the beach topography data for these two areas have been discussed in detail earlier in this section (under “data analysis”).

Analyses of aerial photography of Durban Bluff (Brighton Beach)

The shoreline variation (observed wet-line located at approximately the spring high tide wave runup line) was determined at a cross-section along the Durban Bluff (Brighton) at B10. Aerial photographs covering this area and used in the analysis were available for the years 1937, 1953, 1959, 1968, 1971, 1973, 1978, 1983, 1988, 1990, 1994 and 2003. Using the 1990 vegetation line as basis (i.e. as the fixed reference location), the shoreline variation relative to this 1990 reference line was determined. The results of the study of the variation of the shoreline are summarized in Table 6.8.

Table 6.8: Shoreline variation along the Durban Bluff (Brighton) at B10 from aerial photography.

Year	Shoreline distances (m) relative to reference line	Statistics	Distances (m)
1937	30	Average	18
1953	33	Std. deviation	7
1959	21		
1968	14		
1971	14		
1971	13		
1973	14		
1973	20		
1978	9		
1983	13		
1988	26		
1990	9		
1994	18		
2003	17		

The standard deviation calculated from the aerial photography (Table 6.8) is very similar to that calculated from the topographic surveys (Table 6.1), namely, 7 m and 6.3 m respectively. This is a crucial result, keeping in mind that in the Normal model (Equation 6.1), the key parameter (other than the chosen value of Z), is the calculated standard deviation. Thus, in this case the erosion predicted by means of the Normal model, would yield relatively similar results for the topographic survey data and the aerial photography analysis. For example, for a Z value of 2.33 (99-percentile value), the topographic survey data yields a predicted erosion of 15 m (Table 6.7), while the result based on the aerial photography is 17 m. Thus, in this case at least, the statistical analysis of the shoreline variability based on the aerial photography data, yielded a good prediction of the short-term shoreline erosion that corresponds well with the result based on more extensive and accurate topographic survey data, both using the Normal model.

Analyses of aerial photography of Durban North (outer Bight) at Virginia

In this case, the shoreline variation was measured at three locations along this area (designated Sections I to III). Shoreline variation was quantified by measuring in each case the horizontal distance from the reference line to the high-water runup mark as determined from each aerial photograph. The shoreline variations at Section I to III (based on all the photographs) are shown in Table 6.9.

Table 6.9: Shoreline variation along Durban North (outer Bight) at Virginia from aerial photography.

Shoreline distances (m) relative to landward reference line (2000 orthophoto)				
Year/Section	I	II	III	averages
1937	72	67	257	
1967	59	49	258	
1968	51	41	256	
1983	43	38	259	
1987	49	33	241	
1990	47	56	277	
1997	53	37	260	
2000	51	42	253	
Average	53	45	258	
Std. deviation	8.8	11.3	9.9	10.0

The standard deviations calculated from the aerial photography (Table 6.9) are again very similar to that calculated from the topographic surveys (Table 6.2), namely, averages of 10.0 and 8.5 respectively. Thus, in this case the erosion predicted by means of the Normal model, would also yield relatively similar results for the topographic survey data and the aerial photography analysis. For example, for a Z value of 2.33 (99 percentile value), the topographic survey data yields a predicted erosion of 20 m, while the result based on the aerial photography is 23 m. (Similarly, for a Z value of for example 1.65 (95 percentile value), the topographic survey data yields a predicted erosion of 14 m, while the result based on the aerial photography is 16 m.) Keeping in mind the relatively small number of aerial photographs used in this analysis, the good correspondence of the results is somewhat surprising. Thus, in this case as well, did the statistical analysis of the shoreline variability based on the aerial photography data, yield a good prediction of the short-term shoreline erosion that corresponds well with the result based on more extensive and accurate topographic survey data, both using the Normal model.

Although the results from the two cases that were analyzed were good, this is not yet sufficient evidence to make a robust general conclusion that good aerial photography data can “always” be used to make good predictions of short-term shoreline erosion in South Africa, before further verification is not undertaken with more cases. On the other hand, the greater inaccuracy of the aerial photography data and in some instances relatively few data sets (number of images), will generally result in larger standard deviations. This would result in higher predictions of coastal erosion, as indeed was found to some degree with the two cases investigated here. Such higher erosion predictions mean a more conservative ("safer") erosion setback distance, which is preferable when the uncertainty is greater (or accuracy is less). Such more conservative outcomes may still be totally acceptable when applied with appropriate experience and sound judgment (as indeed should all outcomes be subject to). At least, even if no other data than aerial photography are available, an initial estimate of short-term coastal erosion can be made by judiciously applying the Normal model. There is almost no area along the South African coast for which *at least* 10 different images (aerial photographs and/or satellite images) are not available, which is recommended as the minimum number to provide sufficient confidence in the analysis. Of course, where adequate topographic survey data *is* available, this would almost invariably be the first choice (out of these two data sources: surveys and aerial photography) to use in predicting short-term coastal erosion. In the previous section it was concluded that the Normal model is relatively robust, and provided satisfactory predictions of short-term shoreline erosion under a relatively wide range of conditions. Based on this conclusion and the contention that analysis of aerial photography can provide sufficient data to yield acceptable estimates of short-term shoreline variability virtually anywhere along the South African coast, it can be said that the Norman model provides an apt means of estimating shoreline erosion at beaches all along the South African coast.

6.2.3 A proposed parametric model to determine short-term cross-shore erosion

Background

Many different approaches have been developed to quantify, simulate or predict erosion and accretion of sandy shorelines. These range from predictors only of the direction of net erosion/accretion, to models which quantify local transport rates and time-dependent beach profiles (Schoonees and Theron, 1995). The mainly two-dimensional models which quantify local transport rates and time-dependent beach profiles, as well as the major practical difficulties for application of these models to large study areas, have been discussed previously in Section 6.3.1. Due to such serious difficulties, the actual determination of the short-term cross-shore storm erosion potential along the coastline as assessed for determination of coastal setback lines, is often conducted in an overly simplistic manner. A fixed offset (e.g. 20 m or 40 m erosion setback) is simply assumed along entire sandy study areas (e.g. Cambers, 1997; Houlahan, 1989; Fenster, 2006; Breetzke *et al*, 2012 and Van Weele *et al*, 2013; and WAPC, 2003), which does not account for any other alongshore variation in geo-physical characteristics or coastal processes and dynamics (e.g. wave exposure/shelter, presence or lack of dunes, etc.). Such factors can have a significant effect on the magnitude of cross-shore erosion experienced during sea storms (e.g. Smith *et al*, 2010). Although the assumption of a *conservative*, but well informed fixed offset, can be a robust means of accounting for short-term erosion (or short-term variability and changes) in setback determinations, this requires sufficient historical information and lots of experience, which is not considered satisfactory. Thus, the focus of this section is (rather) on the novel application of basic parameterizations (which to date have only been used to predict the direction of net erosion/accretion), as the basis for developing a one-dimensional model capable of predicting the amount of erosion. The rationale for following this methodology has been discussed in Section 6.2.1.

Assessment of existing predictors of the direction of net erosion/accretion

Seymour and King (1982) evaluated eight predictors of the direction of net cross-shore erosion/accretion, while Seymour and Castel (1988) evaluated another six not previously tested, but found these not to be satisfactory. However, Kraus *et al* (1991) found these previous conclusions to be incorrect. Kraus *et al* examined the capability of basic parameterizations to predict the direction of net beach erosion/accretion due to wave action, emphasizing changes of "engineering interest" such as storm erosion. They found that these parameterizations correctly predicted most erosion events from an extensive field data set which included beaches from around the world. Larson and Kraus (1989) concluded that "even very complex three-dimensional beach changes could be described by one or two non-dimensional parameters" and that if the main processes of beach profile change are identified, the beach response to various wave conditions can be predicted based on semi-empirical relationships. They developed a model aimed at replicating macro-scale beach changes using "standard" data available in most engineering applications. Wright *et al* (1985) developed a successful semi-empirical predictive model of short-term beach changes based on Dean's (1973) heuristic model. (For a further useful discussion of these matters reference is also made to Quick, 1991.)

Based on the literature review and discussion in the foregoing paragraphs, four prediction methods were selected for further evaluation and potential development into parametric models capable of predicting the amount of cross-shore erosion. These were the methods of Dean (1973), Hattori and Kawamata (1980), Sunamura and Horikawa (1974), and Larson and Kraus (1989). These methods are based on the forcing parameters of wave height and period, sediment grain size, various slope terms, deep-water wave length and sediment fall speed, formulated into dimensionless expressions to predict the direction of net cross-shore transport. Sediment fall speed can be directly related to grain size, and deep-water wave length is directly related to wave period, which means that the number of input variables can be further reduced. These relationships are also shown in the following paragraphs.

Dean's 1973 formulation can be written as follows:

$0.80153.H_0/(w.T_p)$ < 1 accretion *(Equation 6.2)*

> 2.5 erosion

With: H_0 = deep water significant wave height

w = sediment fall speed

= $((118920.E6.D_{50}+9398721)^{0.5}-4173)/59460$

D_{50} = median sediment grain size (in m)

T_p = peak wave period

The Hattori and Kawamata (1980) formulation can be written as follows:

$20.944\ .\ H_0^{Mean}.\tan_B/(w.T_p)$ < 1 accretion *(Equation 6.3)*

> 2.3 erosion

With: H_0^{Mean} = $0.625.H_0$

$\tan_B$ = bottom slope to wave break point

The Sunamura and Horikawa (1974) formulation can be written as follows:

$0.205.H_0.(\tan_{20})^{0.27}/(g^{0.33}.(T_p\ .\ D_{50})^{0.67})$ < 1 accretion *(Equation 6.4)*

> 2.0 erosion

With: $\tan_{20}$ = bottom slope to 20 m depth

g = 9.81 (acceleration of gravity constant)

The Larson and Kraus (1989) formulation can be written as follows:

$0.6.L_0\ /\ H_0^{Mean}.(\tan\alpha\ .\ H_0^{Mean}/(w.T_p))^{2.08}$ < 1 accretion *(Equation 6.5)*

> 2.0 erosion

With: L_0 = deep water wave length

= $g\ .\ T_p^2\ /\ (2.\pi)$

$\tan\alpha$ = beach face slope

It should be noted that in the above formulations of Dean, Hattori and Kawamata, and Sunamura and Horikawa, the thresholds have been adjusted from the originals based on the findings of Seymour and Castel (1988). Upon further consideration of the above formulations, it was realized that the Hattori and Kawamata (1980) formulation would be impractical to apply, as it requires the bottom slope to the wave break point, which input data will almost invariably not be available. Thus, the remaining three formulations, namely Dean, Sunamura and Horikawa, and Larson and Kraus (respectively designated Dean, S&H and L&K) were assessed further. The respective sets of formulations were therefore compiled in computer programmes by the author, which were then used to test and develop the methods further.

The rationale behind these predictors is that they should be able to describe the gross cross-shore processes and behaviour of the shoreline based on simplified parameterised functional relationships which reflect the morphologic phenomena on a larger scale (Van Rijn, 1998). The sensitivity of the three formulations to variations of the input parameters was therefore investigated. One parameter was varied at a time while the other parameters were kept constant. The respective constant values for each parameter were: H_0 = 2 m; T_p = 12 s; D_{50} = 0.0006 m (0.6 mm); slope to 20 m depth $\tan_{20}$ = 0.01; beach face slope tan α = 0.1, which are all typical values characteristic of the South African coast. The sensitivity of the three formulations to variations in wave height is show in Figure 6.20.

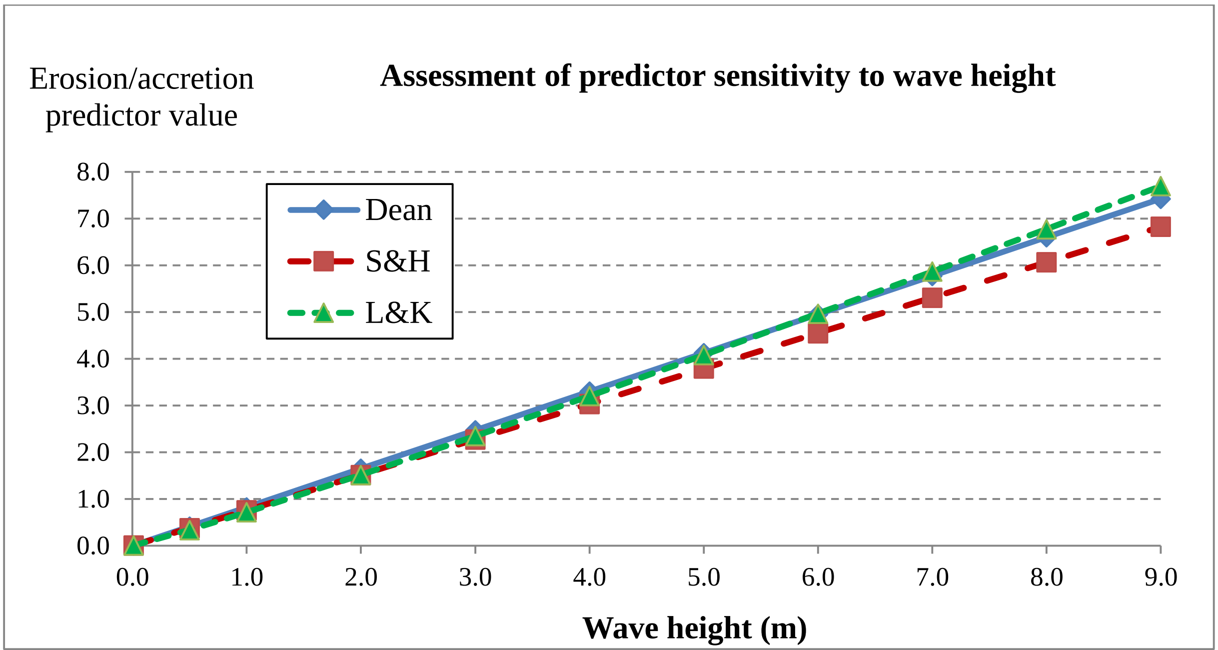


Figure 6.20: Assessment of erosion/accretion predictor response and sensitivity to wave height for Dean (1973), Sunamura and Horikawa (1974; S&H), Larson and Kraus (1989; L&K) predictors

All three formulations exhibit a direct dependency on, and very similar linear response to the wave height. The response is as expected if the basic formulations (Equations 6.2, 6.4 and 6.5) are carefully inspected. This is a satisfactory outcome considering that the main driver of the sediment transport and beach change is deemed to be the waves. Also, all three formulas exhibit the correct general response, in that as the wave heights increase above the average range (in the order of 2.5 m and higher), the predictors show an increasingly strong indication of erosion.

The sensitivity of the three formulations to variations in wave period is shown in Figure 6.21. In accordance with the basic formulation, Dean's formulation shows an inverse relationship with T_p, with diminishing sensitivity as the wave period lengthens. The S&H formulation exhibits a similar relationship but with much reduced sensitivity. The L&K formulation is virtually insensitive to wave period variations.

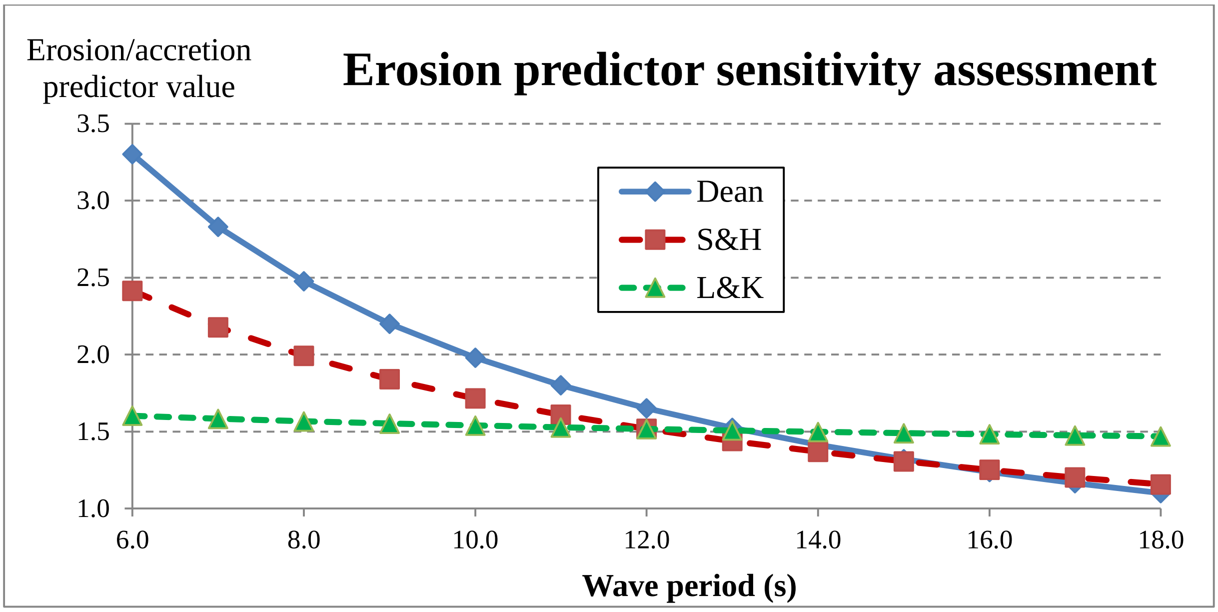


Figure 6.21: Assessment of erosion/accretion predictor response and sensitivity to wave period for Dean, Sunamura and Horikawa (S&H), Larson and Kraus (L&K) predictors

The sensitivity of the three formulations to variations in sediment grain size (D_{50}) is shown in Figure 6.22. All three formulations display an inverse relationship to the grain size with diminishing sensitivity as D_{50} grows, while all three are relatively insensitive to variations in D_{50} beyond about 0.5 mm. S&H's formulation is also relatively insensitive to grain size variations when the D_{50} falls below 0.5 mm. L&K's formulation on the other hand, is extremely sensitive to D_{50} variations, especially when D_{50} lies below say 0.3 mm. Deans's formulation exhibits significant and increasing sensitivity as D_{50} falls below about 0.3 mm, but not extremely so. These markedly different responses of the three formulations to reducing D_{50} values can be attributed to the different exponents of the grain size parameters or fall speeds, as set up in the basic formulations. All three formulations do exhibit the correct general response, in that coarse sediment is less prone to erosion, while fine sands are eroded more easily. Due to the extreme response of the L&K formulation to D_{50} below about 0.25 mm, it is expected that this formulation is probably limited to application where D_{50} is greater than 0.25 mm.

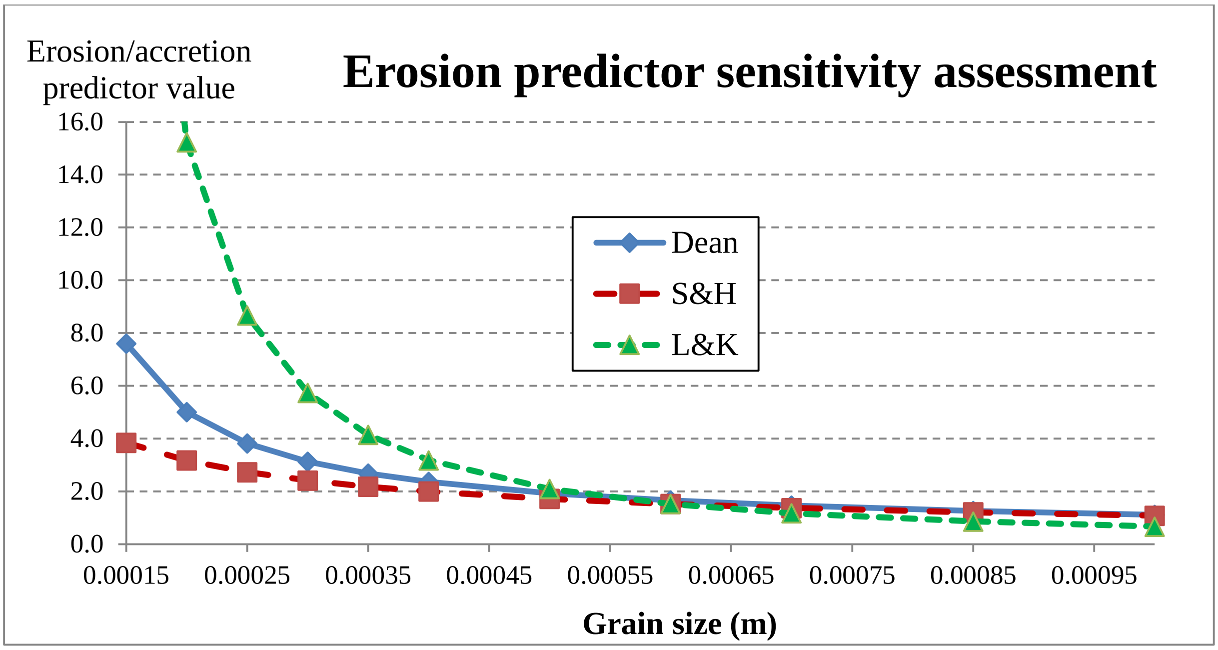


Figure 6.22: Assessment of erosion/accretion predictor response and sensitivity to grain size for Dean, Sunamura and Horikawa (S&H), Larson and Kraus (L&K) predictors

The sensitivity of the three formulations to variations in beach or bottom slope is shown in Figure 6.23. Dean's formulation does not contain a slope term, which is reflected by the horizontal line of its response to slope changes. S&H's formulation shows low sensitivity to slope changes, diminishing further as the slope increases. Keeping in mind that the slope parameter in S&H's formulation is the slope to 20 m water depth, which naturally has a much smaller range (and variability) than the beach slope, this reduced sensitivity appears to be appropriate. The L&K formulation on the other hand, responds strongly to slope changes, with increasing effect as the slope increases (i.e. becomes steeper). This formulation would thus be prone to significant fluctuations due to beach slope fluctuations, which are indeed characteristic of many South African beaches (especially at beach slopes of about 1/10 and steeper).

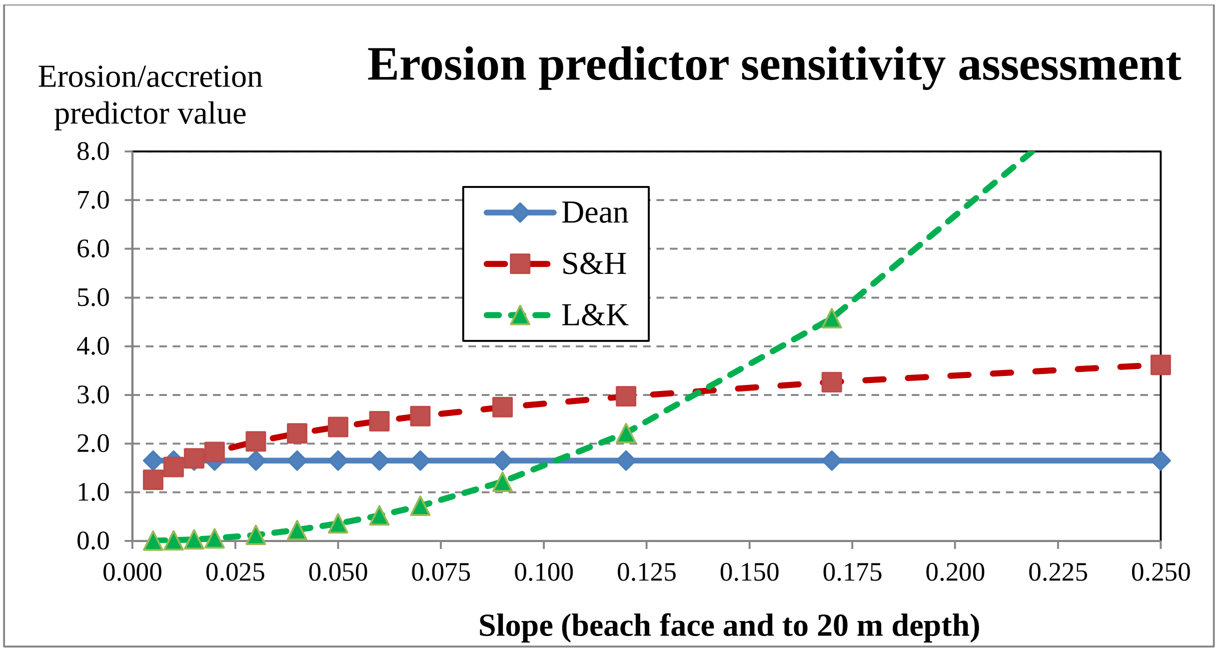


Figure 6.23: Assessment of erosion/accretion predictor response and sensitivity to slope for Dean, Sunamura and Horikawa (S&H), Larson and Kraus (L&K) predictors

Based on the foregoing sensitivity assessment alone, it appears that the L&K formulation would be prone to over sensitivity or have a more limited range of applicability. The sensitivity analyses do not lead to a clear preference regarding the Dean or S&H formulation. However, it is well known that the slope has some influence on sediment transport and shoreline behavior. The lack of a slope term in Dean's formulation therefore leads to an expectation of poorer performance. Despite these concerns, all three formulations were taken forward into further testing for potential development into a model capable of predicting the amount of erosion. This decision was also (partially) influenced by the fact that all three generally exhibited the correct trends in their response to the main drivers.

A proposed parametric model to predict the amount of cross-shore erosion

<u>Beach erosion data</u>

To test the utility of the three formulations for potential development into an erosion prediction model, data was selected from the extensive shoreline monitoring program collected in the vicinity of Durban by the eThekwini Municipality. Five data sets were selected from the exposed Durban Bluff (Brighton) area (Profiles B6 to B10), as well as five data sets from the partially exposed Durban Bight area (Profiles A, D, E, F and 1). These are the same data sets that have been described before in Section 6.2.2, while the locations are indicated in Figures 6.4 and 6.5. The specific data sets were selected on the basis of the good quality of their data (as discussed in more detail in Section 6.2.2), and because they all contained more than 100 individual beach profile recordings (from 106 to 310). These data sets of recorded shoreline variations were analyzed as before (Section 6.2.2) to provide accurate erosion data that could be used to evaluate the erosion predictors. The respective erosion distances for the 50%, 75%, 90%, 95% and 99% percentile values as determined for each of the selected profiles, have been listed in Table 6.5.

Wave height and period data

Wave data was recorded in about 42 m of water depth off Durban over a period of about 7 years. This data (containing some 20 018 records) was analyzed to yield wave height exceedance values as well as the appropriate wave periods linked to these wave heights (CSIR, 1999) . Through a process of reverse shoaling (as described before in Section 5.3), the equivalent deep-sea wave heights were determined. These input wave height and period statistics are summarized in Table 6.10.

Table 6.10: Durban input wave data

	Wave height exceeded				
Percentile	**50**	**25**	**10**	**5**	**1**
Wave height (m) in 42 m depth	1.63	2.01	2.5	2.87	3.96
Deep water wave height (m)	1.78	2.19	2.73	3.13	4.32
Associated wave period (s)	10	11	11	12	12

Sediment grain size and beach slope data

Beach sediment samples are collected along the shorelines in the vicinity of Durban and analyzed by the eThekwini Municipality as part of an extensive shoreline monitoring program. Based on this data, averaged median sediment grain sizes (D_{50}) were calculated for each individual location from the samples collected along the Durban Bight between 1994 to 2012, and similarly along the Bluff (Brighton) from 21 samples collected over 5 years. Averaged beach slopes (tan α; between +1 m to + 3 m CD) were calculated for each individual location along the Bight and Brighton shorelines from the recorded beach profiles (ranging from 106 to 310 profiles each). Average nearshore slopes to 20 m water depth (between 0 m to -20 m CD) in the Bight off each location were calculated from six annual bathymetric surveys of the Bight area. Nearshore slopes to 20 m water depth (between 0 m to -20 m CD) off the Brighton locations were calculated from SAN bathymetric charts. These input data sets on sediment grain sizes, beach slopes and slopes to 20 m water depth are summarized in Table 6.11.

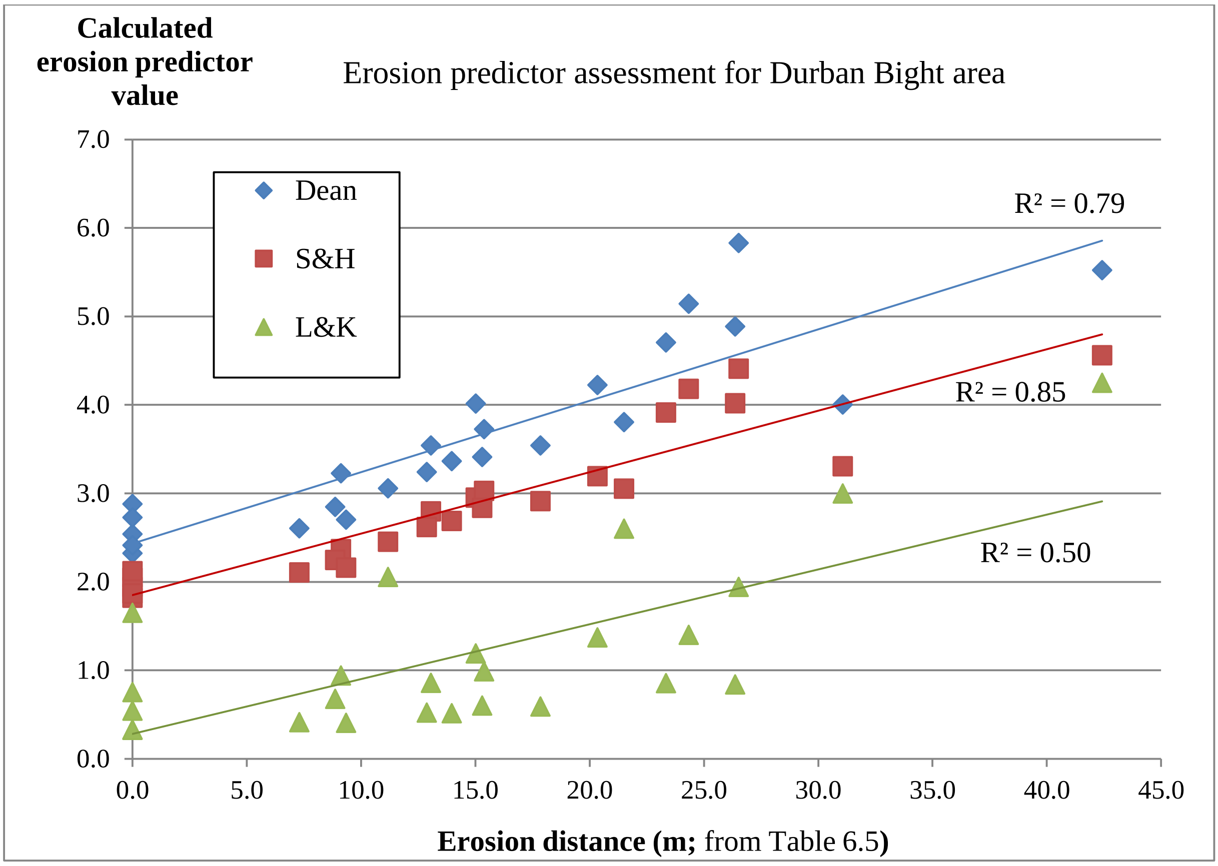
Table 6.11: Durban input data - sediment grain sizes, beach slopes and slopes to 20 m depth

Durban Bight station numbers	**A**	**D**	**E**	**F**	**1**
Average D50 values (mm)	0.367	0.395	0.435	0.418	0.347
Average beach face slope	0.071	0.045	0.039	0.037	0.046
Slope to 20 m water depth	0.0100	0.0087	0.0087	0.0086	0.0077
Brighton (Bluff) station numbers	**BR6**	**BR7**	**BR8**	**BR9**	**BR10**
Average D50 values (mm)	0.585	0.613	0.758	0.674	0.641
Average beach face slope	0.110	0.102	0.122	0.115	0.103
Slope to 20 m water depth	0.02	0.02	0.02	0.02	0.02

Correlating erosion predictors and erosion distance

Based on the three formulations of Dean, Sunamura and Horikawa, and Larson and Kraus (Equations 6.2, 6.4 and 6.5), and the values of the input parameters listed in Tables 6.10 and 6.11, erosion predictor values were calculated for each individual location and wave condition. The erosion predictor values were calculated for each specific wave condition, thus for wave height exceedance percentile values at 50%, 25%, 10%, 5% and 1% (with the associated wave periods). These erosion predictor values were then correlated to the appropriate erosion distances percentile data values at 50%, 75%, 90%, 95% and 99% respectively from Table 6.5. In other words, the predicted and data values are “inversely” related, meaning that, for example, the 25% erosion predictor value (calculated from the 25% wave height exceedance values) is correlated to the 75% erosion data value, or similarly the 10% erosion predictor value (calculated from the 10% wave height exceedance values) is correlated to the 90% erosion data value. The reason for relating the predictions and data is this seemingly inverse manner, just follows from how the erosion data was collated. As explained more fully in Section 6.2.2 and Table 6.5, the erosion distance percentile data was calculated such that, for example, the 75% value (a relatively small erosion distance) means that 25% of the recorded erosion distances were greater than this value, while the 99% value (a large erosion distance) means that only 1% of the recorded erosion distances were greater than this value. Accordingly, the calculated erosion predictor values for the 50%, 25%, 10%, 5% and 1% wave height exceedance values need to be correlated to the erosion distances percentile data values (Table 6.5) at 50%, 75%, 90%, 95% and 99% respectively. The results for the Bight and Brighton areas are shown in Figures 6.24 and 6.25 respectively.

It may be observed that none of the three formulations tested here account for storm duration. It was mentioned in Section 6.2.1 that the duration of the event can also affect the amount of erosion that occurs (e.g. Bosom and Jimenez, 2011, and Callaghan *et al*, 2008), which means that a storm with a lower wave height but longer duration, could lead to a similar amount of erosion due to a storm with larger waves but shorter duration. However, in terms of the wave parameters, the method developed here (as discussed further in this section) correlates the wave height (exceedance values) to the erosion distances (equivalent exceedance values) based on longer term statistical data without considering the storm durations. This is in line with the stated rationale to develop robust methods that require the minimum amount of input data, but accounting for storm duration is an aspect that could be investigated in future work to potentially increase the accuracy of the method.



Figures 6.24: Erosion predictor assessment for Durban Bight area

In the Durban Bight area, the Dean and S&H predictors exhibited clear trends consistent with the erosion data (Figure 6.24), with relatively strong correlations (R^2 values of 0.79 and 0.85 respectively). The L&K predictor exhibited much more scatter here and did not correlate very well to the data (R^2 = 0.5). The poor performance of the L&K predictor against the Durban Bight data is ascribed to the mild beach slopes of the Bight profiles (tan α ranged from 0.037 to 0.071), and the over sensitivity of this predictor to such low slope values (as observed in Figure 6.23). In the Brighton area (Durban Bluff), all three predictors exhibited clear trends consistent with the erosion data (Figure 6.25), with relatively strong correlations (R^2 values of 0.75, 0.79 and 0.85 for the Dean, S&H and L&K predictors respectively).

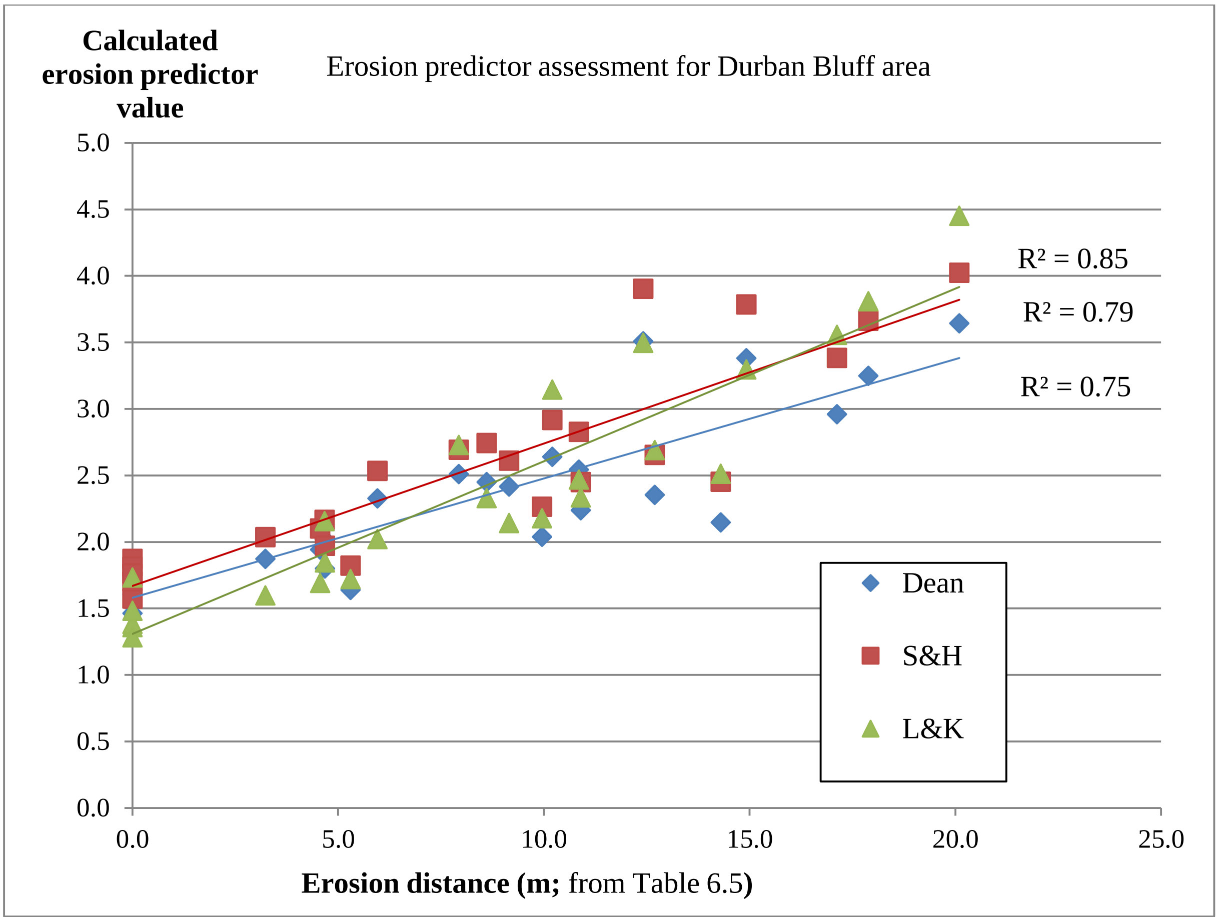


Figure 6.25: Erosion predictor assessment for Brighton (Durban Bluff) area

<u>A proposed parametric model to determine short-term cross-shore erosion</u>

In the foregoing it was shown that all three predictors exhibited clear trends consistent with the erosion data, indicating that functional relationships for predicting the amount of erosion are feasible. Based on the assessment of the correlations between the erosion predictors and recorded erosion distances, the S&H formulation gave the best overall performance (average R^2 = 0.82), followed by the Dean formulation, while the L&K formulation was unsatisfactory in some instances. This is also consistent with the sensitivity assessment where it was found that L&K formulation would be prone to over sensitivity, contributing to the decision to reject the L&K formulation. It was also decided not to pursue Dean's formulation further, because of the before mentioned expectation of its poorer performance due to the lack of a slope term, and the fact that overall it fared second best in terms of the correlation.

Thus, based on the correlations found in Figures 6.24 and 6.25, a relationship was formulated on the S&H parameterization to predict the amount of erosion. A linear relationship (based on the method of least squares) was fitted through the combined data of both the Brighton and Durban Bight of recorded erosion distances. The fit of this relationship through the data is indicated in Figure 6.26. As expected from the previous evaluations, the trend displays the correct response to the drivers. Although a few outliers are observed, the scatter of the data about the line is generally relatively small ($R^2 = 0.79$), and the fit is apparently acceptable. It should be noted that the line does not pass through the origin, in that predictor values of less than about 1.5, yield a negative erosion distance. This is correct, as a negative erosion distance actually means accretion, indicating that as the predictor value falls below about 1.5, an increasing likelihood of accretion is predicted. However, the intention is not to predict the amount of accretion (in fact the small amount of erosion predicted (< 5 m) for predictor values of between 1.5 and 2 is certainly less than the accuracy of the method). The derived formulation for predicting cross-shore erosion distance (E, in m) is as follows:

$$E = A \,.\, P_{SH} - C \qquad \textit{(Equation 6.6)}$$

With $$P_{SH} = 0.205\,.H_0.(\tan_{20})^{0.27}/(g^{0.33}.(T_p\,.\,D_{50})^{0.67})$$

A is a dimensionless coefficient, which can be said to be indicative of the rate (or ”sensitivity”) of erosion response, and has here empirically been determined to have a value of 10.6 (based on the least squares fit of predicted to recorded data as described in the foregoing paragraph and indicated in Figure 6.26). C is also a dimensionless coefficient (which can be interpreted as a “response modifier” shifting the boundary of net erosion/accretion response), and has here empirically been determined to have a value of 16 (determined in the same way as A). Equation 6.6 can be described as a one-dimensional parametric shoreline erosion model, with the input parameters being the offshore wave height and period (H_0 and T_p), the sediment grain size (D_{50}) and the bottom slope to 20 m depth ($\tan_{20}$).

The predicted erosion distances based on this model (Equation 6.6) were now directly compared to the recorded erosion data (combined Brighton and Durban Bight data) as indicated in Figure 6.27. Although the scatter of the predictions about the 1-to-1 line (100% accurate) remains significant, the general performance of the model is acceptable.

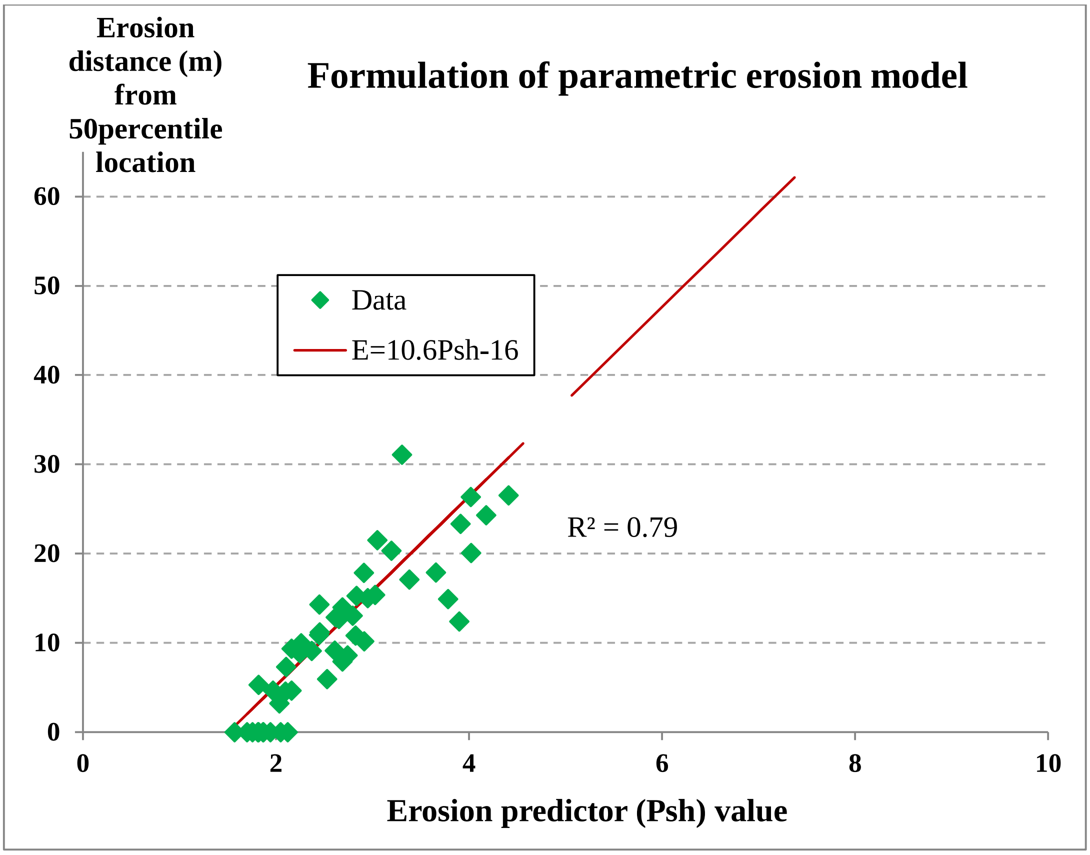


Figure 6.26: Fit of the erosion formulation through the erosion data.

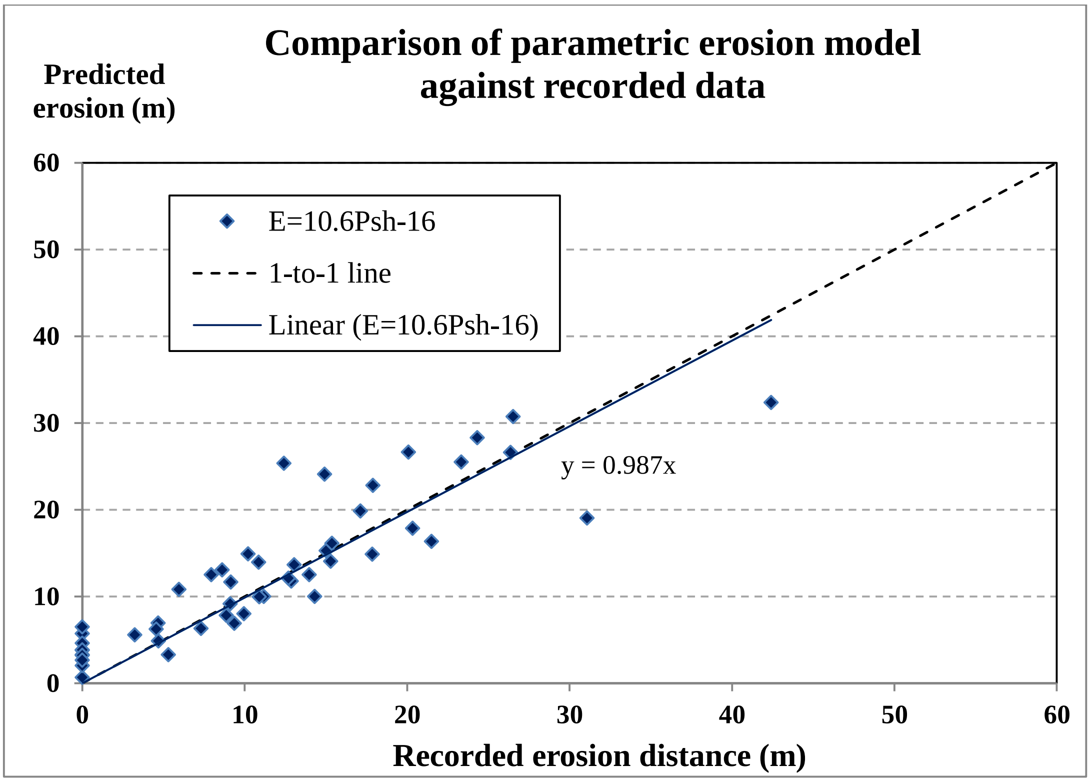


Figure 6.27: Comparison of predicted erosion (Equation 6.6) to the recorded erosion

Validation of Parametric Model

To validate the satisfactory performance of the model, additional model tests were performed on new data from two different areas not used in the derivation of this model.

Exposed location - Virginia Beach

For the first model validation, four profiles were selected from the exposed Virginia Beach area located to the north of Durban. This is the same erosion data as described before in Section 6.2.2 (containing about 95 recordings of shoreline position at each location). The wave input data has also been described before (Table 6.10). Averaged median sediment grain sizes (D_{50}) were calculated for each individual location from samples collected between 2007 and 2012 by the eThekwini Municipality. Nearshore slopes to 20 m water depth off the Virginia locations were calculated from SAN bathymetric charts.

The same procedure as described before was followed to analyze the recorded erosion data and to predict erosion with the Parametric model. The erosion amounts predicted with the Parametric model (Equation 6.6) are compared to the recorded erosion distances in Figure 6.28. Again it is clear that the predicted erosions compare reasonably well with the data, with relatively small scatter of the prediction around the 1-to-1 line (100% accurate), and the model under-predicting by only a very small amount (about 1%) on average.

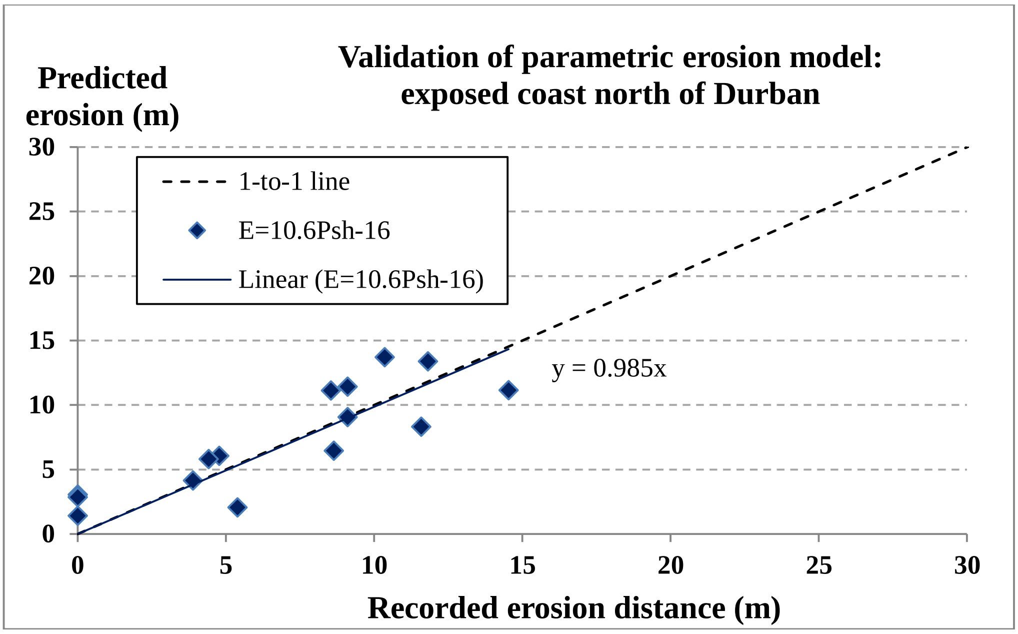


Figure 6.28: Parametric model validation for an exposed coastal area located to the north of Durban

Sheltered location - inner Durban Bight

A further validation of the Parametric model was conducted to test whether the model is also applicable in even more sheltered areas, as the previous tests were conducted in areas ranging from exposed open coast to partially sheltered locations. For this purpose, a profile was selected which is located about midway along the southern (inner) Durban Bight shoreline. The same data sources and analyses procedure, as described above were followed. The erosion amounts predicted with the Parametric model (Equation 6.6) are compared to the recorded erosion distances in Figure 6.29. In this case the model clearly performed very well with accurate predictions and very low scatter. On average the model slightly over-predicted (by about 4%).

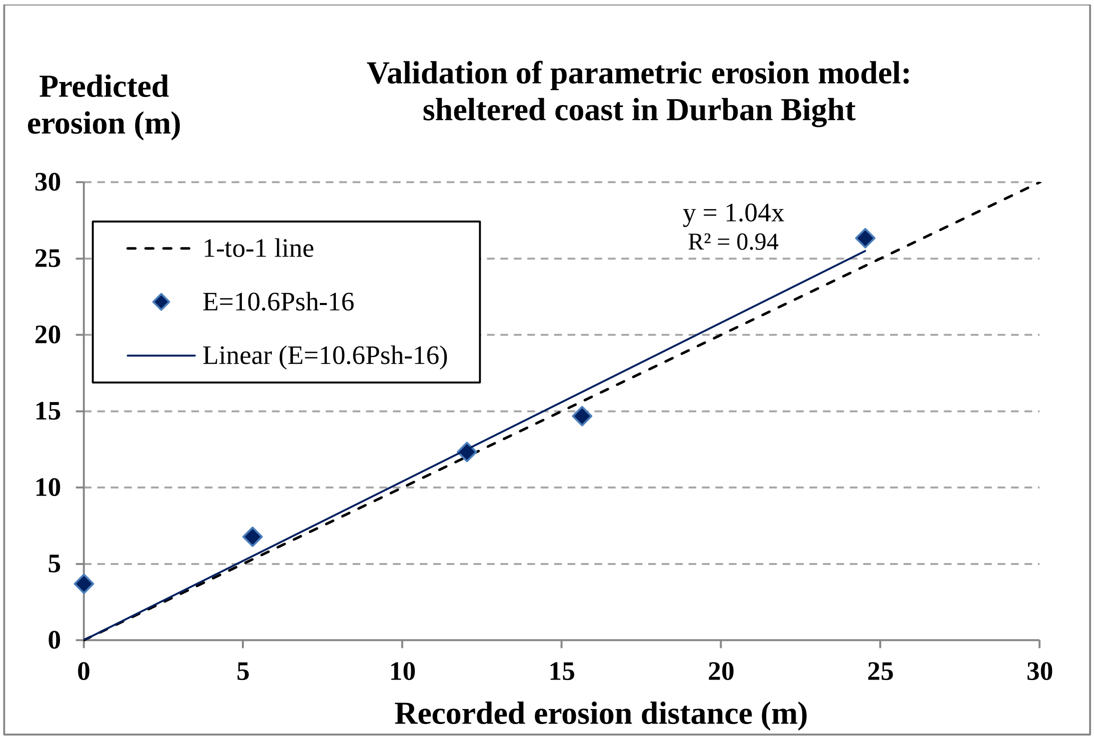


Figure 6.29: Validation of parametric erosion model: sheltered coast in Durban Bight

The foregoing satisfactory results attest to the veracity of the model and lead to the conclusion that the parametric erosion model is relatively robust, providing satisfactory results under a relatively wide range of conditions (Table 6.12). Ideally the model should be tested further against additional data from other South African areas, but at this stage, there do not appear to be other South African sites where sufficient data has been recorded to allow robust comparisons to be made. (To calculate accurate percentile values from recorded shoreline erosion data for rigorous testing of the models, 100 or more surveyed profiles are ideally required, although reasonably accurate percentiles up to 90% value can be calculated from about 50 or more recorded beach profiles, depending also on the representativeness of the data.) Yet, based on the foregoing, the model is expected to be generally applicable for use in South Africa. A final verification of the model is conducted in the following section (Section 6.2.4), by applying the model to two new study areas and comparing the results with those of a comprehensive (i.e. process based approach) model, as well as to the Normal model.

Table 6.12: Ranges of the parameters used to calibrate and verify the Parametric model (Equation 6.6)

Input parameter	Minimum value	Maximum value
Deep water significant wave height (m)	1.78	4.32
Peak wave period (s)	10.0	12.0
Median sediment grain size (mm)	0.347	0.83
Slope to 20 m water depth	0.0068	0.022

6.2.4 Application and comparison of the proposed erosion prediction models

Additional case studies were conducted to compare the Normal (Equation 6.1) and Parametric (Equation 6.6) erosion prediction methods and further test their abilities by selecting two diverse new study areas, namely Richards Bay and Table Bay. Thereby, and by further comparing the results from these two models to those of a more complex modern erosion model (i.e. with a much stronger theoretical basis), namely the Van Rijn (2008) model, their robustness and applicability for a variety of environments that are representative of the South African coast are assessed. (The Van Rijn model is discussed in Section 6.2.1.) Erosion predictions were made for return periods ranging from 1-in-1 year to 1-in-100 years.

Richards Bay

Both of the new models, as well as the Van Rijn (2008) model, were applied to the same study area, namely the area covered by profiles 1 to 6 listed in Table 6.4. The slope, grain size and shoreline variation data was determined for this area as described before in Section 6.2.2. The predicted erosion results for the three models are indicated in Figure 6.30. The results from all three models compare relatively well, with the differences in predicted erosion distances ranging from 0 m to a maximum of 13.2 m for the 1-in-100 year return period. This good result from a new study area and good correspondence with a more complex modern model further heightens confidence in the validity of the two new models.

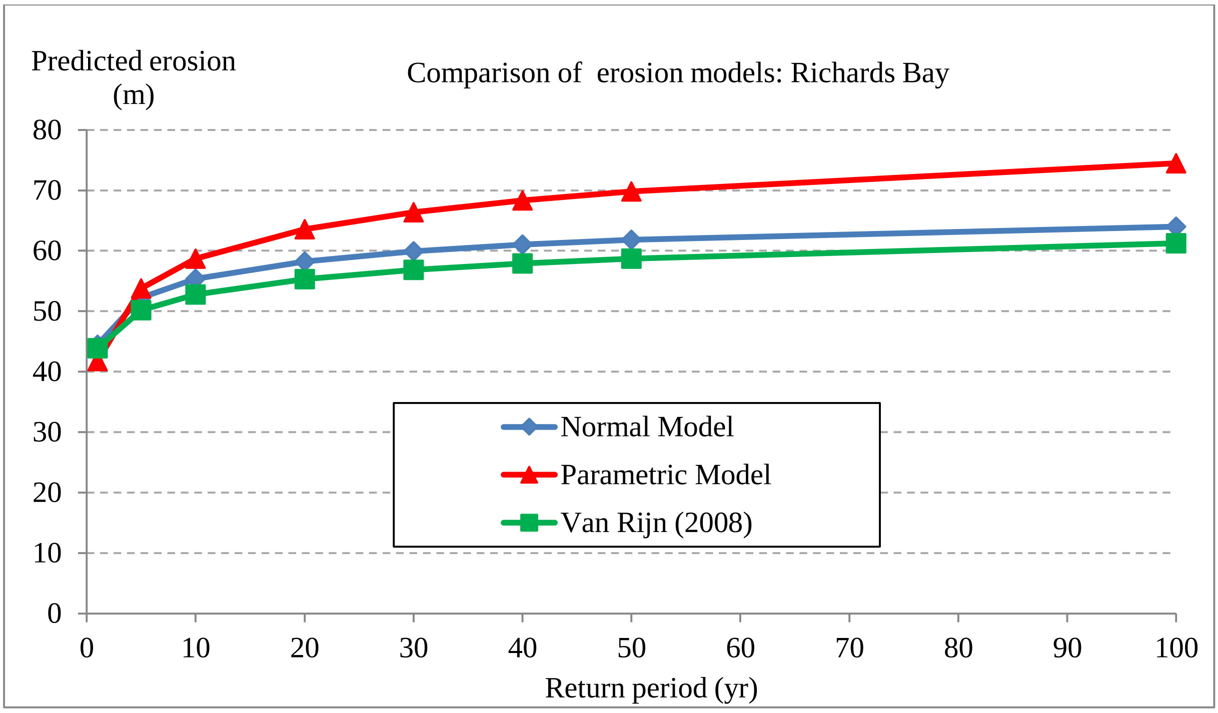


Figure 6.30: Comparison of the Normal, Parametric and Van Rijn models for return periods of 1-in-1 year to 1-in-100 year at Richards Bay.

Table Bay

Following the same procedure as before, both of the new models, as well as the Van Rijn (2008) model, were applied to a new study area, namely the area covered by the Table Bay wave runup studies as discussed in Section 5.3. The slope, grain size, water levels and wave data were determined for this area as described before in Section 5.3. The predicted erosion results for the three models are indicated in Figure 6.30. The results from all three models compare very well, with the differences in predicted erosion distances ranging from 0 m to a maximum of only 4.4 m for the 1-in-100 year return period. DEAD & P (2010) conducted SWAN wave modelling in conjunction with SBEACH modelling (Larson and Kraus, 1989) to simulate storm erosion of the Table Bay shoreline (as reviewed in Section 2.3.2) for the same area as considered here. Their SBEACH cross-shore morphological modelling yielded an erosion prediction of 33 m for the 100 year return period case. This means that the results from all four models, the Normal, Parametric, Van Rijn and SBEACH models, lie within a narrow total band of only 7 m, which implies excellent correspondence and bolsters the credibility of the two new models.

Setback lines are determined for pre-selected planning horizons, which typically range from 50 years to 200 years or more (the selection of appropriate planning horizons is discussed in detail in Section 8.1). This implies that future conditions due to potential climate change effects also need to be considered. To illustrate how the potential effects of climate change could be incorporated into the erosion predictions, an assumed scenario of a 10% across the board increase in extreme wave heights was simulated by means of the Parametric model. The results are also indicated in Figure 6.31 (the open square purple symbols and line), and are in accordance with the expected response based on the formulation of the Parametric model (Equation 6.6). Thus, a simple climate change scenario (in terms of wave height) could easily be incorporated into the predicted erosion based on the Parametric model.

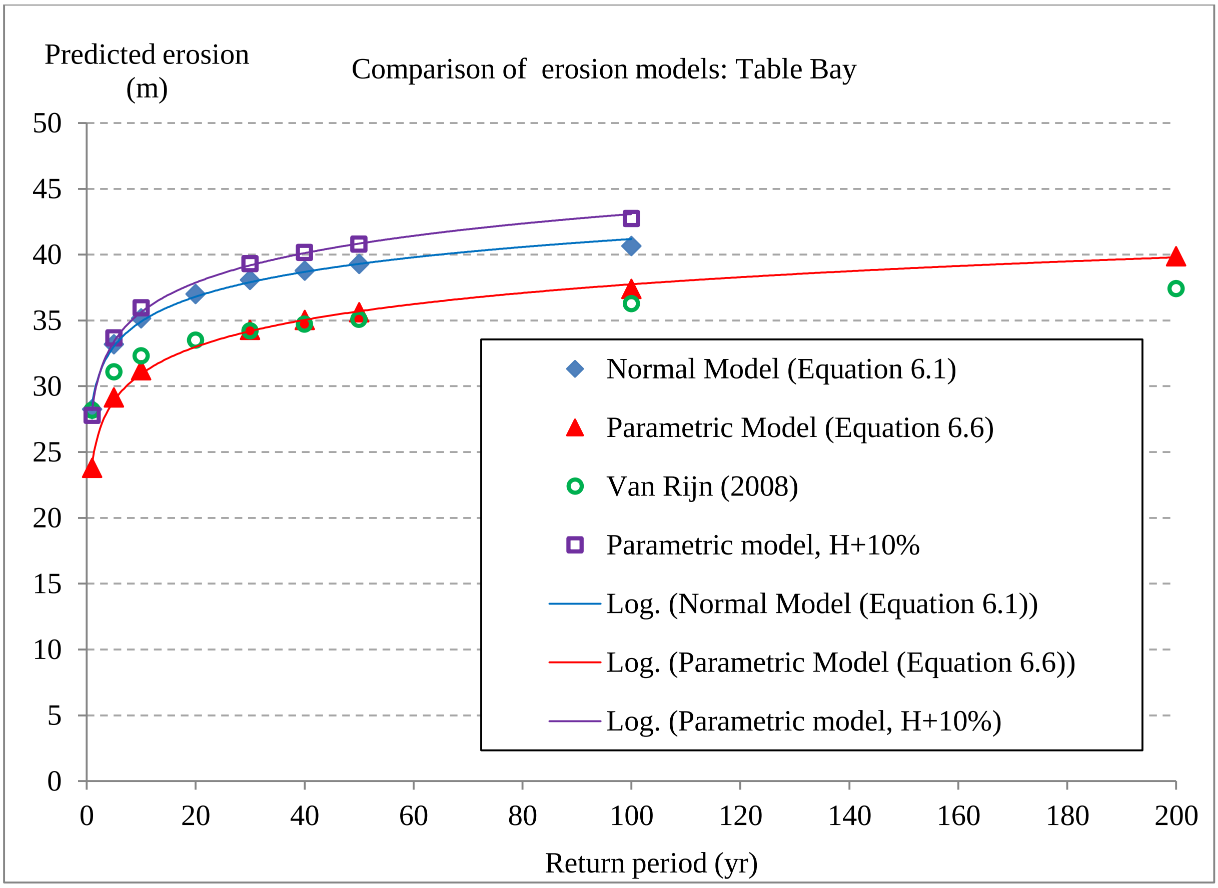


Figure 6.31: Comparison of the Normal, Parametric and Van Rijn models for various return periods at Table Bay.

As mentioned, setback lines can be determined for a range of planning horizons. Most of the erosion predictions depicted in Figure 6.31 have been determined for storms (extreme wave heights) with return periods up to 100 years (i.e. considering time scales of up to 100 years). To illustrate how the potential erosion predictions can be made for longer recurrence intervals, wave heights were calculated for a 200 year return period (based on a statistical extrapolation of the extreme wave heights). Erosion predictions were then made for these 1-in-200 year wave heights by means of both the Parametric and Van Rijn models. The results have been added to Figure 6.31 (the red triangles and green circles for the 200 year return period), and in this case these two models still yield very similar results even at the long time scale of 200 years. Thus, different return periods (in terms of wave height) can easily be incorporated into the predicted erosion based on the Parametric model.

The good results achieved with the Normal and Parametric models for two diverse new study areas and good correlation in both instances with a more complex modern model, in addition to the testing and validations described in Sections 6.2.2 and 6.2.3, are considered to have sufficiently established the verification of the two new erosion models for general application in South Africa (and potentially even wider). Certainly the two types of sandy coasts that occur most commonly in South Africa (as characterised in Sections 2.2 and 2.8) are well represented by these case studies and the test sites used in Sections 6.2.2 and 6.2.3.

6.3 Long-term shoreline changes and trends

6.3.1 Accounting for observed long-term shoreline location trends

On a dynamically stable coast, the shoreline erosion is mainly a function of the beach characteristics and the severity of the wave attack resulting in shorter term fluctuations where average erosion and accretion are balanced in the longer term. However, in some instances, shoreline stability is also affected by progressive long-term erosion or deposition patterns (resulting in long-term "instability"). Long-term progressive erosion trends may be attributed to many different direct or indirect causes (Section 6.1), often without the specific effect of a particular cause being well quantified. Yet, it is critical to identify any long-term erosional trends, such as for example mentioned in Section 6.2.2 regarding some areas near Richards Bay (Figure 6.16). A continuing long-term erosional trend of about 1 m per year means that on average the shoreline would retreat by 30 m over the next 30 years, for example. If the beach is currently say about 50 m wide (on average), that would mean that the remaining beach width would theoretically be only 20 m after a period of 30 years. However, short-term erosion due to sea storms may result in more erosion than 20 m in one event, which in this example would mean that the backshore area (perhaps facilities or the primary dune) would be subject to direct wave attack and possible under scouring. This example is given to illustrate why an assessment of long-term stability or possible erosional trends is so important. Yet, this remains an aspect that is often overlooked.

Long-term shoreline changes can be quantified by considering the variation in shoreline location over an extended period. Vertical aerial photography is a useful aid for doing this. (According to O'Connel (2003) the wet-line observed in aerial photography can be used to give a good approximation of the shoreline location along sandy areas at the time that an aerial photograph was taken.) Smith and Zarillo (1989) have shown that the total error in measurements of long-term change in shoreline position from aerial photographs can be as much as 10 m. It must be stressed that the distortions that may occur in aerial photographs and satellite images have to be removed as far as possible by geo-referencing (for example, to a high quality undistorted ortho-photo). Comparative beach surveys also provide a good indication of the stability of the coastline. Topographic beach survey data is much more accurate than aerial photographic data. However, aerial photographs often span much longer time periods than existing beach surveys and provide a longer term perspective. Aerial photographs are, therefore, especially useful in identifying long-term trends, while beach surveys provide more accurate information on shorter term variability (Section 6.2.2).

If a significant eroding trend is apparent in the shoreline location (e.g. from analyses of aerial photography), a conservative estimate of the erosion rate is extrapolated for the stipulated/chosen setback “design lifetime” or ”return period”, usually 50 or 100 years, and then added to the erosion setback line. Making provision for a long-term trend therefore safeguards coastal developments against future projected recession over the stipulated or chosen setback “design lifetime, say 50 or 100 years of progressive erosion.

The following example is provided to illustrate how such provision (additional setback distance) is determined where a long-term eroding trend is found (in this case along the coast to the south of Durban). In this analysis, twelve sets of aerial photographs were used to determine the shoreline location (at approximately the spring high tide wave runup line). Thus, the historic shoreline locations relative to a fixed reference line were determined, as shown in Figure 6.32. A significant long-term eroding trend in the shoreline location is apparent and this beach has retreated by about 13 m since 1937. The average regression is about 0.23 m/year, which means, for example, that an additional setback distance of 12 m should be provided for a 50 year planning horizon (or 7 m for a 30 year period). The assumption is thus made that that linear extrapolation of the historic trend calculated over the total record is representative of the expected future situation (i.e. the underlying cause of the shoreline regression will continue to have the same net effect in future). If there is good reason or evidence to expect a modified future shoreline response, this should be incorporated, which could simply be done by, for example, fitting a curve to the historical data with an appropriate accelerating or decelerating erosion rate rather than just a linear extrapolation. (In this particular example, it may be said that the shoreline appears to have been "dynamically stable" since about 1960, as seen in Figure 6.32. Thus it could be argued that in this particular case, there is not sufficient justification to assume an ongoing eroding trend based on the total recession over the whole data period. It should further be kept in mind that much of the observed changes are within the accuracy of aerial photographic analysis methods, and also within the natural variability of shorelines along the East Coast. The point here is not whether there is truly an ongoing erosional trend at this particular site, but to illustrate the principle of accounting for long-term trends.)

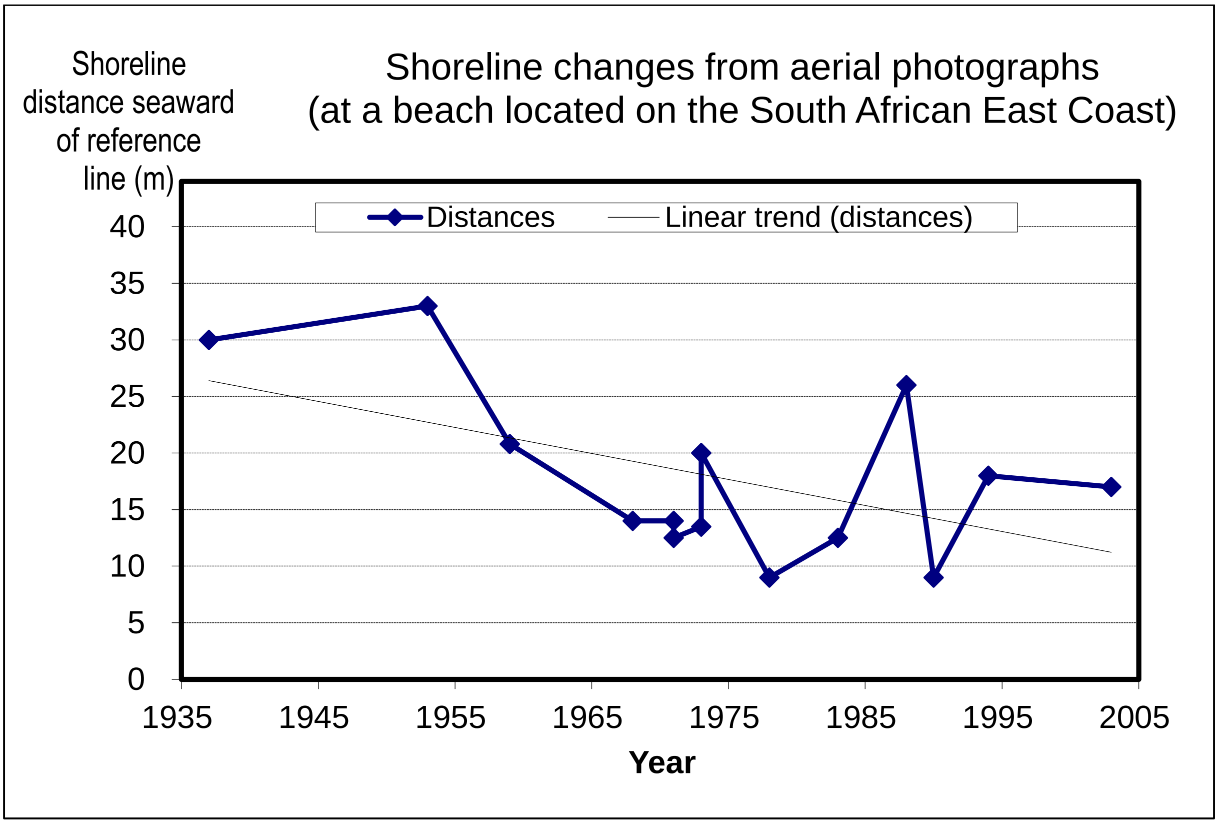


Figure 6.32: Long-term shoreline location changes from aerial photography (at a beach located on the South African East Coast).

The aforementioned procedure for calculating the additional provision required (additional setback distance) where a long-term eroding trend is observed is therefore relatively straightforward and usually robust in terms of inputs needed for determination of setback lines.

6.3.2 Long-term shoreline change modelling

The direct projection method described in the preceding Section 6.3.1 is suitable where future shoreline evolution (longer term changes) is expected to be mainly driven by the same influences that resulted in past recorded changes (over the preceding 30 to 100 years). (Geologic processes that have formed the shoreline over pervious centuries and millennia are largely beyond the time scales considered in planning or managing coastal development, infrastructure and setback lines.) However, where future shoreline evolution will mainly be driven by anthropogenic influences, for example coastal and marine structures or sand pumping or artificial beach nourishment, the most appropriate method is to apply numerical modelling. Such modelling can account for various future scenarios (changing anthropogenic influences), while the analyses of past in-situ shoreline behaviour obviously (usually) does not account for such future changes. Where the long-term shoreline evolution is mainly affected by gradients in alongshore sediment transport or changes in sand supply, for example due to beach nourishment, the most appropriate tool is usually shoreline modelling.

Line models, which represent the nearshore bathymetry by one or more depth contour lines, are a relatively simple method of predicting nearshore bathymetry changes as a function of time. In a one-line model, the beach or shoreline change is usually represented by just the horizontal (or planform) movement of the shoreline (say 0 m MSL) or for example the +2 m MSL contour. At present the most commonly used one-line models are the GENESIS (Hanson, 1989) and UNIBEST (UNIform BEach Sediment Transport; Delft Hydraulics, 1994, 2005; Deltares, 2013) models. In these models, the beach profile is assumed constant, on-offshore sand movements are neglected (i.e. considered to be in long-term equilibrium with no net on/offshore movement within the modelled littoral cell), and all shoreline changes are associated with gradients in the longshore transport rate (Hanson and Kraus, 2011). Larson *et al* (1987, 1990) describe the basic approach taken, where a control volume of sand is considered and the mass balance during an infinitesimal time interval is established. The local sediment balance at the coast is based on the continuity equation which is rewritten to represent the effect of the angle of wave attack on the magnitude of sediment transport (Bosboom and Stive, 2012). The longshore sediment transport rate can be determined by various applicable formula, for example the CERC formula (US Army Corps of Engineers, 1984), the Kamphuis (Kamphuis, 1991, and modified Kamphuis method; Mil-Homens *et al*, 2013), the Van Rijn (2002, 2014) models, etc. In the one-line model, the instantaneous position of the shoreline is then a function of the active profile height, the angle of wave attack and the rotation of the shoreline (Hugo, 2013).

A major input required for such modelling is the nearshore wave climate along the study area, which usually means that mathematical wave modelling needs to be conducted using a state-of-the-art refraction model. Other important inputs required are the beach profile, the closure depth and active height of the profile, and the sediment parameters.

The profile closure depth is defined as that depth below the water surface at which no significant changes in the profile occur over a specified time period. This time period is usually taken as equivalent to the period over which shoreline changes are being modelled, which typically ranges from 10 years to 50 years, but could be as short as in the order of one year. If available bathymetric survey data is too sparse to determine the closure depth (as determined by analysing the recorded changes between different bathymetric surveys conducted over several years), it can be estimated by theoretical means. Based on the nearshore wave data and the sediment characteristics, the profile closure depth and active height of the profile can be determined, by using the theoretical methods of Swart (1974) or Hallermeier (1981) and Birkemeier (1985). According to Birkemeier (1985), the annual profile closure depth, (d_1), can be calculated relative to the mean low water level as:

$$d_1 = 1.75 \,.\, H_e - 57.9 \left(H_e^2 / (g \,.\, T_e^2)\right)$$

where H_e is the nearshore significant wave height exceeded 12 hours per year, T_e is the associated wave period and g is the gravitational acceleration.

Another critical requirement is that like every other model (physical or mathematical), the one-line model must also be calibrated based on the recorded response of the geophysical coastal environment. Thus, the one-line model should be calibrated and verified against existing patterns of shoreline changes within the study area.

In South Africa one-line modelling (of shoreline evolution) has been applied with success at numerous locations, for example, Oranjemund (Soltau *et al*, 2002), Richards Bay (Swart, 1981; Coppoolse *et al*, 1994; Luger *et al*, 2002), Port Elizabeth (CSIR, 1995), Elizabeth Bay (Smith *et al*, 2002), Alexander Bay (Theron *et al*, 2003), and Mossel Bay (Hugo, 2013). Despite these successful local applications, it is important to keep the purpose and limitations of this kind of modelling in mind.

Limitations of one-line shoreline modelling are in practice mainly related to these models being one-dimensional ("1-D"). This means that two- and three-dimensional processes such as wave-driven undertow and vertical variations in sediment concentration are not resolved. This is usually not a significant problem, since the sediment transport in typical application study areas is dominated by horizontal gradients, e.g. the longshore transport due to oblique wave attack. However, rip and eddy currents around structures (e.g. breakwaters and groynes) are also not accounted for in the 1-D models. Shoreline changes predicted in the vicinity of structures (within say one to two wavelengths from the head or tip of obstacles) do not properly account for localised diffraction effects. Thus, shoreline predictions directly in the lee (down-drift) of structures are not reliable. The cross-shore component of the sediment transport, including erosion during storm events, is also not simulated. (The effect of cross-shore phenomena can be assessed with, for example, the Unibest-TC and Unibest-DE modules (Deltares, 2013) or other cross-shore models as described in Section 6.2.1.) More complex situations, where both cross- and longshore flows or sediment transport patterns are affected by human interference, may therefore require more sophisticated two- or three-dimensional hydrodynamic, sediment transport and morphologic modelling (such as the Delft3D suite of models; Deltares, 2011a, 2011b or XBeach; Roelvink *et al*, 2009). Such two- or three-dimensional modelling is only suitable for detailed investigations of relatively small study areas where simulations are limited to relatively short periods (usually not more than a few years). Although some "acceleration" schemes (e.g. schematizations and/or increased morphologic time-step schemes) have been developed that may in specific circumstances be applied to enable longer term simulations (e.g. De Vriend *et al*, 1993; Hanson *et al*, 2003; Ranashinghe *et al*, 2011), the two- or three- dimensional modelling is mostly unsuitable for prediction of specifically long-term shoreline evolution.

Ultimately, it can be said that, in general, one-line modelling (of shoreline evolution) is a robust well established tool-of-the-trade of coastal engineering practise that has also been applied with success at numerous locations in South Africa, and therefore further research of this technology is not required for such applications in terms of setback lines. It should also be kept in mind that for the purposes of determining setback lines, one-line modelling would only really be appropriate in those limited circumstances where future shoreline evolution will mainly be driven by anthropogenic influences, for example sand pumping and artificial beach nourishment or coastal and marine structures.

6.3.3 Prediction of the planform of curved beaches

Curved beaches are normally found along bays between headlands. These beaches have been called crenulated-shaped beaches, half-hart beaches or headland-bay beaches (Hsu *et al*, 1989). Theories exist which can be used to predict the planform of these beaches. Hsu and Evans (1989) derived a so-called parabolic bay shape equation for this purpose (see also CEM, 2004). According to Hsu *et al*

(1989), the shoreline orientation at the downdrift side of a parabolic bay in equilibrium is parallel to the wave crests (that is, the wave incident angle), as indicated in Figure 6.33. Further developments include: (1) a modification of the Hsu and Evans (1989) method by Gonzales and Medina (2001); and (2) the use of MEPBAY software (Klein *et al*, 2003) to visually evaluate the existing shoreline in relation to the static equilibrium planform of the beach, which has been applied with success to natural and man-made Spanish beaches. A significant problem in applying such models is the difficulty in determining the control points (e.g. the headland diffraction point) in a robust manner. Lausman *et al* (2010a; 2010b) published useful findings towards addressing this problem to some degree.

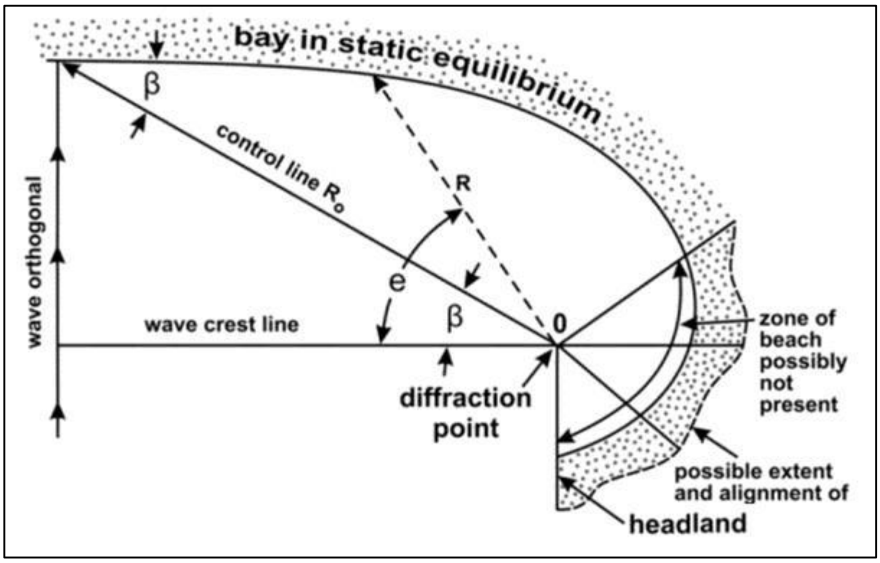


Figure 6.33: Definition sketch of Hsu and Evans' (1989) parabolic bay shape model *(from Hugo, 2013).*

The wave incident angle is an important parameter when predicting the equilibrium form of a parabolic bay, emphasising the need for accurate input data for design. For example, the maximum indentation of a parabolic bay can typically increase by between 2.5 % and 10 % if the wave incident angle increases by 5°, according to the theory of Hsu *et al* (1989). This means that the model can be used to give an indication of the relatively shorter term shoreline planform shifts associated with significant wave direction changes (e.g. if the wave climate exhibits a significant seasonal change in direction). The model is usually employed to investigate potential long-term situations: for example where the developed shoreline (i.e. the planform as adjusted to anthropogenic activities) might be different from the natural equilibrium bay shape; or mostly, to get an estimate of the maximum indentation (i.e. erosion or setback) expected due to extreme scenarios or long-term shifts in forcing conditions.

Such methods can thus be used to predict the theoretical planform of curved beaches, including the maximum indented planform, (i.e. most eroded bay planform expected). This then gives an indication of where the erosion setback line could be located. The model is best applied where wave incident angles are limited to distinct narrow bands, such as for example found in smaller inner bays or pocket beaches located in sheltered locations (within larger bays). However, where the wave regime is highly variable, especially in terms of the incident wave angles, the application of such models becomes more difficult and the results are more indicative than accurate. Such conditions are characteristic of most of the South African coast and although informative, the indicative results achieved are not readily compatible with the robust results required for setting setback lines. Such methods (e.g. parabolic bay shape models) therefore have limited application in South Africa, and are not considered further in terms of suitable methods which need to be widely applicable for determining setback lines along our coast.

6.4 Conclusions

Shoreline changes and coastal erosion (both long- and short-term) are a key component of all coastal setback lines and also a major focus of this thesis. Many different approaches have been developed to quantify, simulate or predict erosion and accretion of sandy shorelines. The conventional cross-shore sediment transport or morphology models quantify local transport rates and time-dependent beach profiles, but are typically “data-hungry” (or reliant on inputs from other models) and require significant calibration. In complex situations (for example, where both cross- and longshore hydrodynamic processes drive shoreline behavior, or where sediment transport patterns and beach morphology are affected by human interference), the more sophisticated two- or three-dimensional hydrodynamic, sediment transport and morphologic modelling are typically required. Such two- or three-dimensional modelling is mainly suitable for detailed investigations of relatively small study areas and face major practical difficulties essentially preventing application to large study areas.

To overcome these difficulties associated with applying conventional (2D or 3D) modelling and the accompanying need for extensive (and expensive) data collection and calibration, alternative approaches were pursued. Thus, new methods were developed and two alternative approaches are proposed, requiring less input data to quantify short-term shoreline erosion, and that are also suitable for larger scale approaches.

The premise for the statistical approach, is that the erosion setback related to cross-shore shoreline variations can be determined if it can be shown that the variations follow a known statistical distribution, and the parameters of this distribution have been quantified. Based on extensive analyses of numerous individual locations (53) situated in three different coastal regions, and exhibiting a large variety of characteristics, it is concluded that the short-term shoreline variation of protected, moderately protected, and exposed, natural beaches in South Africa is mostly normally distributed. This can be used to predict the maximum landward movement over a selected period (say 50 years), based on the Normal model (Equation 6.1). To test the veracity of this method of predicting erosion, high quality long-term data sets were selected from two areas that are quite different in nature, with individual profiles having wide-ranging characteristics. The Normal model provided satisfactory results under a fairly wide range of conditions and it appears that the model is relatively robust. To validate the satisfactory performance of the Normal model, additional tests were conducted on new sites located in areas which differ from the two areas where the model was first tested. Again it was clear that the predicted erosions compared very well with the data.

For much of the South African coast, topographic survey data is not available, thus the utility of aerial photography data for the purpose of predicting erosion was investigated. In the test areas, the statistical analysis of the shoreline variability based on the aerial photography data, yielded good predictions of the short-term shoreline erosion that correspond well with the results based on more extensive and accurate topographic survey data, both using the Normal model. At least, even if no shoreline change data other than aerial photography is available, an initial estimate of short-term coastal erosion can be made by judiciously applying the Normal model. There should be virtually no area along the South African coast for which at least 10 different images are not available, which is recommended as the minimum number to provide sufficient confidence in the analysis. In determining setback lines, it is in any case critical to identify and quantify any long-term shoreline erosion trends (as discussed in Section 6.3.1). Therefore such analysis of aerial photography is usually required anyway, meaning that the input data for application of the Normal model would already be generated and would not require additional work to produce.

This method (using the Normal model) can be considered more realistic than many others, in that it is based on actual measured shoreline behaviour of the study area. Relatively little additional data or measurements are required and no “calibration” of complex theories or models is needed. The required analyses of topographic beach surveys and/or imagery such as aerial photography is not onerous and would be conducted anyway as part of good practice in determining setback lines (as discussed further in Chapter 9). Application of the Normal model also constitutes a relatively simple and robust method. Thus, use of this method is recommended in most circumstances.

The focus of the second approach was on the novel application of basic parameterizations, as the basis for developing a model capable of predicting the amount of erosion. The rationale behind developing these predictors is that they should be able to describe the gross cross-shore processes and behavior of the shoreline based on simplified parameterised functional relationships which reflect the morphologic phenomena on a larger scale (Van Rijn, 1998), thus also enabling efficient application in large study areas. Following initial investigations, three formulations were taken forward into further testing for potential development into a model capable of predicting the amount of erosion. To test the utility of the three formulations, data was selected from an extensive shoreline monitoring program. Based on the assessment of the correlations between the erosion predictors and recorded erosion distances, the Sunamura and Horikawa (S&H) formulation gave the best overall performance, and a relationship was formulated on the S&H parameterization to predict the amount of erosion. The derived formulation for predicting cross-shore erosion distance is given in Equation 6.6. This can be described as a one-dimensional parametric shoreline erosion model, with the input parameters being only the offshore wave height and period (H_0 and T_p), the sediment grain size (D_{50}) and the bottom slope to 20 m depth (tan_{20}), thus having modest input data requirements. The predicted erosion distances based on the Parametric model were now directly compared to the recorded erosion data. The general performance of the model is apparently acceptable. To validate this conclusion, additional model tests were performed on new data from two different areas not used in the derivation of this model. It is clear that the predicted erosions compare reasonably well with the data. A final validation of the Parametric model was conducted to test whether the model is also applicable in even more sheltered areas, as the previous tests were conducted in areas ranging from exposed open coast to partially sheltered locations. The model clearly performed very well with accurate predictions and very low scatter. The satisfactory results attest to the veracity of the model and lead to the conclusion that the Parametric erosion model is relatively robust, providing satisfactory results under a relatively wide range of conditions, and also enables efficient application in large study areas.

Finally, additional case studies were conducted to compare the Normal and Parametric erosion prediction methods and test their abilities by selecting two diverse new study areas, as well as by further comparing the results from these two models to those of a more complex modern erosion model (i.e. with a much stronger theoretical basis). The good results from the new study areas and good correspondence in both instances with the more complex modern model, in addition to the testing and validations described in Sections 6.2.2 and 6.2.3, are consirered to have sufficiently established the verification of the two new erosion models, attesting to their robustness and applicability for a variety of environments that are representative of the South African coast (and potentially even wider). Certainly the two types of sandy coasts that occur most commonly in South Africa (as characterised in Sections 2.2 and 2.8) are well represented by these case studies and the various test sites used in Sections 6.2.2 and 6.2.3.

In determining setbacks it is also critical to identify any long-term shoreline erosion trends, yet this remains a feature that is often neglected. An example is given to illustrate why an assessment of long-term stability or possible erosional trends is so important. Long-term shoreline changes can be quantified by considering the variation in shoreline location over an extended period. Vertical aerial photography is especially useful in identifying long-term trends, while beach surveys provide more accurate information on shorter term variability. If a significant eroding trend is apparent in the shoreline location, a conservative estimate of the erosion rate is extrapolated for the stipulated/chosen setback ”return” or “planning period”, usually 50 or 100 years, and then added to the erosion setback line. This direct projection method is suitable where future shoreline evolution is expected to be mainly driven by the same influences that resulted in past recorded changes. Where the long-term shoreline evolution is mainly affected by gradients in alongshore sediment transport or changes in sand supply, for example due to beach nourishment, the most appropriate tool is usually shoreline modelling. A major input required for such modelling is the nearshore wave climate along the study area, which usually means that mathematical wave modelling needs to be conducted using a state-of-the-art refraction model. Other important inputs required are the beach profile, the closure depth and active height of the profile, and the sediment parameters. The shoreline model should be calibrated and verified against existing patterns of shoreline changes within the study area. In general, 1D shoreline modelling is robust and well established, and in those limited circumstances where such modelling to simulate shoreline evolution is warranted, it is more than adequate for applications regarding setback lines in South Africa.

Based on the foregoing conclusions, appropriate, sufficiently robust and defendable methods have indeed been found and developed to efficiently predict coastal erosion in large study areas (both long- and short-term) in a “data poor” environment. In conjunction, recommendations are provided for practical and implementable methodologies to determine the coastal erosion/recession components of setback lines in South Africa.

Chapter 7: A method to account for the additional protection provided by dunes in determining setback lines

In Chapter 2 it was found that current methods of determining setback lines have not adequately taken dune effects into account. This chapter is therefore aimed at developing a technical remedy for this specific shortcoming in the methods that have been applied to determine setback lines in South Africa. In view on the thesis objectives, the method should be appropriate for "data poor" environments, suitable for efficient application in large study areas, but still be sufficiently robust and defendable.

7.1. Introduction

7.1.1 Introduction and rationale

Historically, encroachment by development has threatened and destroyed many coastal dunes (e.g., Figure 7.1). This is problematic as the dunes serve not only as a natural asset for biodiversity but also form an important coastal defense system. In some instances, it is only the maintenance of the primary dune that provides ongoing protection to coastal infrastructure against wave attack and erosion. Coastal developments and unmanaged public access to beaches and dunes can cause dune erosion, which will reduce the dune volume and associated natural coastal protection.

Figure 7.1: Example of dune crest removed and partially vegetated with exotic grass to provide a sea view *(Photo A Theron)*

Reduced coastal erosion is expected in regions where the beach is backed by a dune, as dunes contribute significantly to reducing short-term storm erosion and retarding recession. This effect has been noted by many authors, for example Vellinga (1986), Steetzel (1993b), Theron *et al* (2010a) and Barwell (2011). Yet, current methods of determining setback lines have not adequately taken dune effects into account. This chapter is therefore aimed at developing a practical method to account for the additional protection provided by dunes in determining setback lines in South Africa.

Dunes are often located relatively near to the sea and then have a direct effect on shoreline response during most cross-shore dynamics associated with wave action, thus having an active effect within a typical envelope of recorded shoreline locations. However, it is also not uncommon for a dune to be located further landward, beyond the envelope of recorded shoreline locations (data typically spanning a few years to less than 30 years in South Africa), but still within the expected extreme (e.g. one in 50 year) setback distance (Figure 7.2). Such a more "distant" (landward) dune would have had virtually no effect on the recorded shoreline variability, yet it will provide additional protection against more extreme erosion events expected to occur within the next say 50 years. Thus, the additional "erosion reduction effects" of such dunes would not have featured in the recorded shoreline response on which most methods or models to predict erosion, would have been based or calibrated. Such dune effects have therefore not adequately been taken into account in the methods most often applied to date. Consequently, this chapter is focused on developing a method to account for the added protection provided by dunes in determining setback lines. The principle of this novel “dune” method is to reduce predicted erosion retreat by accounting for the effect of dunes that contain large volumes of sand which assist in arresting erosion.

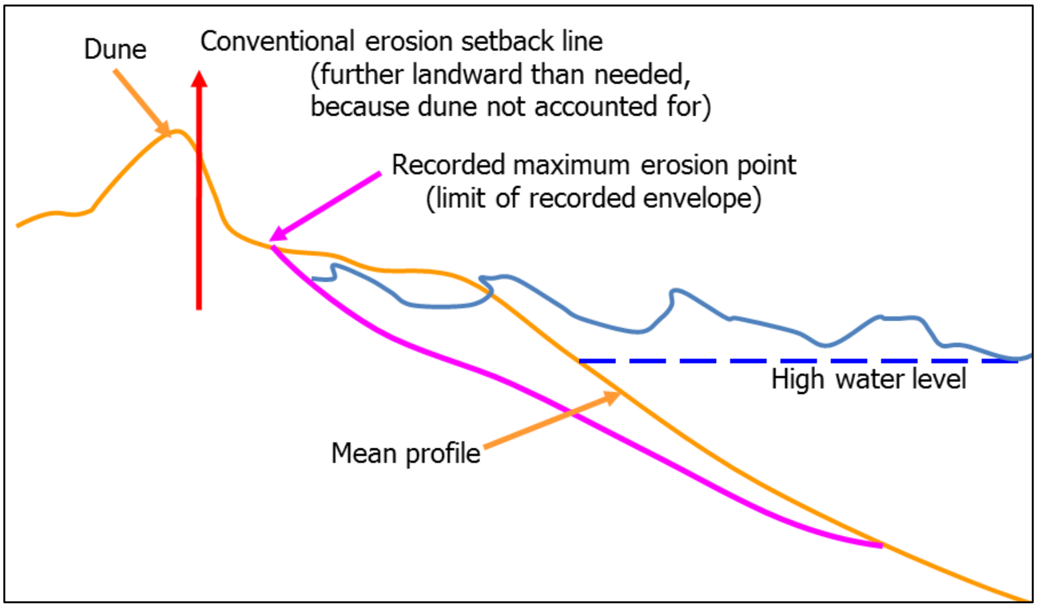


Figure 7.2: Schematic diagram of a dune located landward of recorded shoreline envelope, but within predicted erosion setback

7.1.2. Beach and dune morphology models

Schoonees and Theron (1995) evaluated 10 of the most well-known time-dependant cross-shore sediment transport/morphological models. Of the 6 models with dune capabilities, four were classified into the “acceptable” or “better” groups in terms of both their theoretical basis and the extent to which they were verified. These four models were the Bailard, Durosta, Watanabe and SBEACH models. (For full descriptions of these models, see Schoonees and Theron (1995), which includes the primary references.)

The present literature review found that the models used most widely for "dune" applications appear to be the SBEACH (Larson and Kraus, 1989; Larson *et al*, 1990), Durosta (Steetzel 1987, 1993a, 1993b) and EDUNE (Kriebel, 1995) models, while the Vellinga (Vellinga 1982, 1986) model was used more in the past. Taking into account more recent developments, the Durosta model has been incorporated into UNIBEST (Delft Hydraulics, 1994, 2005) as UNIBEST-DE (Dune Erosion), the C2SHORE model developed by US Army WES (Johnson and Grzegorzewski 2011; Grzegorzewski *et* *al* 2013) has extended the previous CSHORE model (Kobayashi *et al*, 2009), while the XBeach model (hurricane impacts within the Morphos model system) has been developed by Roelvink *et al* (2009). (In UNIBEST-DE, the net local cross-shore transport rate is still derived from the depth integrated product of the time-averaged velocity and sediment concentration profiles, as per the Steetzel model (Steetzel, 1993a).) The UNIBEST-DE and XBeach models are possibly the best of the five more modern models, noting that the suitability of the C2SHORE (and its predecessor CSHORE) model could not be sufficiently ascertained from available literature. In general, it can be said that the more modern (and complex) the model is, the larger is the computational effort required to run the model, the more input data (or outputs from other numerical models, for example, wave models) is required, and the more field data is required to verify and calibrate the model. The CEM (US Army Coastal Engineering Research, 2004, Part IV-4-5 p66) advises that SBEACH remains a very useful model, but requires experience in practical application (as should be the practice for virtually all numerical models).

Concentrating on the SBEACH cross-shore transport/morphological model, it was developed to predict beach profile change resulting from cross-shore sand transport, focusing on the main morphologic features of bars and berms (Larson and Kraus, 1989; Larson *et al*, 1990). Many of the assumptions and relationships used in developing the model are founded on measurements made during large wave tank tests. Changes in the beach profile are assumed to be caused by breaking waves. Therefore, the cross-shore transport rate is determined from the local wave, water-level, and beach profile properties; and the equation describing the conservation of beach material is solved to compute profile change as a function of time. Although SBEACH does not simulate the longer term onshore sand transport processes associated with post-storm beach recovery, this is not a shortcoming in terms of its application here (as discussed further in the following sections of this chapter). Thus, SBEACH has an acceptable theoretical basis and has been most extensively verified. (In addition, it can simulate dune overwash.) Finally, the SBEACH model is widely and freely available. In contrast, the UNIBEST model (Delft Hydraulics, 1994, 2005) requires a relatively expensive commercial license, while the EDUNE (Kriebel, 1995) and C2SHORE (Johnson and Grzegorzewski 2011) models are not easily obtainable (the suitability of the C2SHORE model is also unknown). The XBeach model might theoretically be the best, but requires substantial computational effort, and much input data (outputs from other models).

Based on all of the above, it was concluded that the SBEACH model would be a good tool to investigate the effects of dunes on shoreline erosion in relation to setback line studies. As expected, the literature review further showed that dune volume would be a useful parameter to consider (Steetzel, 1993b; Barwell, 2011; and others), and that it was more applicable than dune height in terms of its effect on erosion.

7.1.3. Approach

Following from Section 7.1.2, it was decided to use the SBEACH model to try and find a practical and robust relationship between dune volume and setback distance, applicable to the South African coastline. Thus, extensive modelling was conducted for three diverse sites: one at Richards Bay in northern Kwazulu_Natal on the Indian Ocean seaboard; another at Walvis Bay on the Atlantic Ocean seaboard along the southwestern African coast; and the third at two Western Cape sites, one of which is a semi-shelterd location within Table Bay, while the other is a more exposed site to the north of Cape Town. (The SBEACH model has been calibrated and applied for these sites before, and gave reasonable results [Schoonees and Theron, 1996; Schoonees *et al* 1998; WSP, 2008; DEAD & P, 2010].) The input data required in SBEACH for modelling the effects of storms is: the initial beach profile, median sand grain size, wave characteristics and a time series of the water-levels

7.2. Test cases

7.2.1 Richards Bay test cases

A view of this study area in provided in Figure 7.3.

Figure 7.3: View of the Richards Bay test case site *(Photo A Theron)*

<u>The main input parameters were as follows:</u>

Waves:	Storm of September to October 1990 (H_s max = 6.1 m)
	Wave heights and periods as recorded by a buoy in 22 m water depth
Real time modelled:	41 days (time step = 6 min.)
Water levels:	Predicted levels from the Naval tide tables
	(The wave setup is calculated within SBEACH. Being an open coast location, wind effects were considered to be low and were therefore not taken into account.)
Median grain size:	0.3 mm
Reference profile:	Averaged from available beach and bathymetric surveys

To quantify what effect different dune configurations and locations would have on shoreline erosion, 13 different configurations were added onto the reference profile. The pre-calibrated model was then used to simulate the profile response to the same input time series (the October 1990 storm as described above). Thus, the predicted shoreline response for each configuration could be compared to the predicted erosion for the reference profile (as surveyed).

<u>The input profiles (dune scenarios) simulated for comparison of "dune versus no dune" effects are as follows:</u>

- Peaked dunes with heights of 3 m, 5 m, 7.5 m and 10 m (an example is shown in Figure 7.4);
- Flat topped dunes with heights of 5 m and 10 m;
- These dunes were placed in 2 different locations: landward of the + 2 m MSL (mean sea level) and landward of the + 3.1 m MSL contours;
- "Sandy bluff" type dune with a height of 7.5 m (i.e. the dune crest continues landward at a fixed height);
- Deflated profile with the area above + 2.5 m MSL removed (i.e. no dune); and
- Deflated profile with the area above + 3 m MSL removed (i.e. no dune).

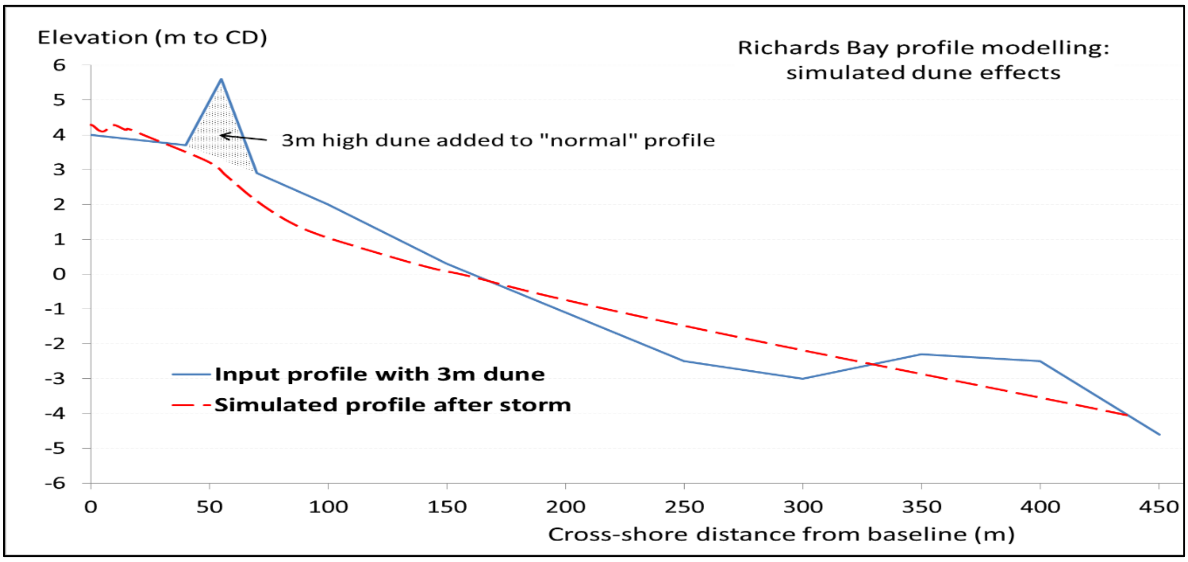


Figure 7.4: Richards Bay modelling of dune effect, example with a simulated 3 m high peaked dune

<u>The main results were as follows:</u>

The main output from the model is the predicted profile response (for example, Figure 7.4). Figure 7.4 shows the input profile (solid blue line) with a 3 m high dune (indicated by the grey area) added onto the reference profile (in this specific case), as well as the predicted output profile (dashed red line) after the effects of the storm have been simulated. For this investigation the most relevant output was the predicted horizontal erosion at the +2 m MSL contour (near the high water mark). Thus, the predicted erosion of the reference profile (as surveyed) due to the storm of October 1990 is 21 m. The predicted erosions for the other input profiles (various dune configurations as listed above) range from 0 m to 25 m. (Further interpretation of the results is discussed after the next test case which follows.)

7.2.2 Walvis Bay test cases

A view of this study area in provided in Figure 7.5.

***Figure 7.5: View of the coastline in the vicinity of the Walvis Bay test case site** (January 2012; Photo A Theron)*

The main input parameters were as follows:

Waves:	Constant wave height of 4 m (H_0s) which is just less than the 1-in-10 year storm. Design wave period (T_p) = 12.5 s.
Real time modelled:	41 hours (average duration) (time step = 2 min.)
Water levels:	Predicted levels from the Naval tide tables (varying from -0.04 m CD to +1.79 m CD)
Median grain size:	0.35 mm (average of eight samples)
Reference profile:	According to the beach and bathymetric survey

Similar to the previous Richards Bay test procedure, 14 different configurations were added onto the reference profile, to quantify what effect different dune configurations and locations would have on shoreline erosion. The pre-calibrated model was then again used to simulate the profile response to the same input time series (the "medium" storm as described above). Thus, as before, the predicted shoreline response for each configuration could be compared to the predicted erosion for the reference profile.

The input profiles (dune scenarios) simulated for comparison of "dune versus no dune" effects are as follows:

- peaked dunes with heights = 1.5 m; 3 m; 7 m
- flat topped dunes with heights = 5 m; 10 m
- "sandy bluff" type dunes with heights = 5 m; 7.5 m (i.e. the dune crest continues landward at a fixed height; an example is shown in Figure 7.6.)
- deflated profile with area above + 2 m MSL removed (i.e. no dune)
- deflated profile with area above + 2.5 m MSL removed (i.e. no dune)

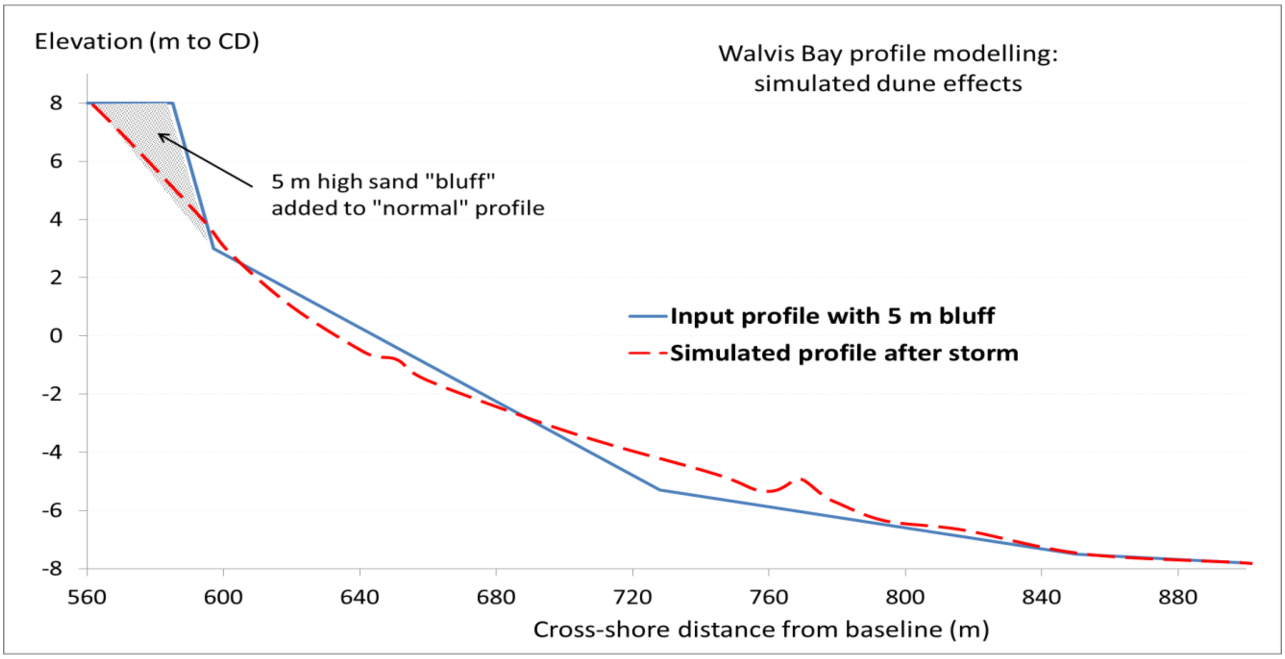


Figure 7.6: Walvis Bay modelling of dune effect, example with a simulated 5 m high bluff (+3 to +8m CD)

The main results were as follows:

The same procedure as described before for the Richards Bay test was employed here. The predicted erosion of the reference profile (as surveyed) due to the 4 m storm waves is 18 m (at +2 m MSL). The predicted erosions for the other input profiles (various dune configurations as listed above) range from 0 to 30 m (for example Figure 7.6). Further interpretation of the results is discussed in the following section.

7.2.3 Cape Town test cases

<u>The main input parameters were as follows:</u>

Waves:	Storm of 30 August – 1 September 2008 (H_{mo70} max = 10.7 m; T_p = 18.3 s) Wave conditions in 21 m water depth as modelled from buoy recordings in 70 m water depth.
Real time modelled:	3 days (30 August – 1 September, time step = 3 min.)
Water levels:	Recorded levels from nearby Port of Cape Town (These recorded levels were 0.3 m higher than the predicted tidal levels; thus a water level setup of about 0.3 m was experienced within the port. Based on wind data recorded in Table Bay, an *additional* wind setup of 0.2 m was calculated for the site in Table Bay outside of the port (procedure as per Coastal Engineering Manual [US Army Coastal Engineering Research, 2004]).
Median grain size:	0.24 mm (Table Bay) and 0.32 mm (Melkbos)
Reference profile:	According to the topographic beach surveys and available bathymetric charts

Similar to the previous Richards Bay and Walvis Bay test procedure, 8 different configurations were added onto the reference profile, to quantify what effect different dune configurations and locations would have on shoreline erosion. The SBEACH model was then again used to simulate the profile response to the same input time series (the "30 August – 1 September 2008" storm as described above). Thus, as before, the predicted shoreline response for each configuration could be compared to the predicted erosion for the reference profile.

<u>The input profiles (dune scenarios) simulated for comparison of "dune versus no dune" effects are as follows:</u>

- peaked dunes with heights = 4 m (an example is shown in Figure 7.7.)
- "sandy bluff" type dunes with heights = ranging from 4 m to 11 m (i.e. the dune crest continues landward at a fixed height; an example with a 4 m height is shown in Figure 7.8.)
- deflated profiles with area above + 3 m MSL removed (i.e. no dune)

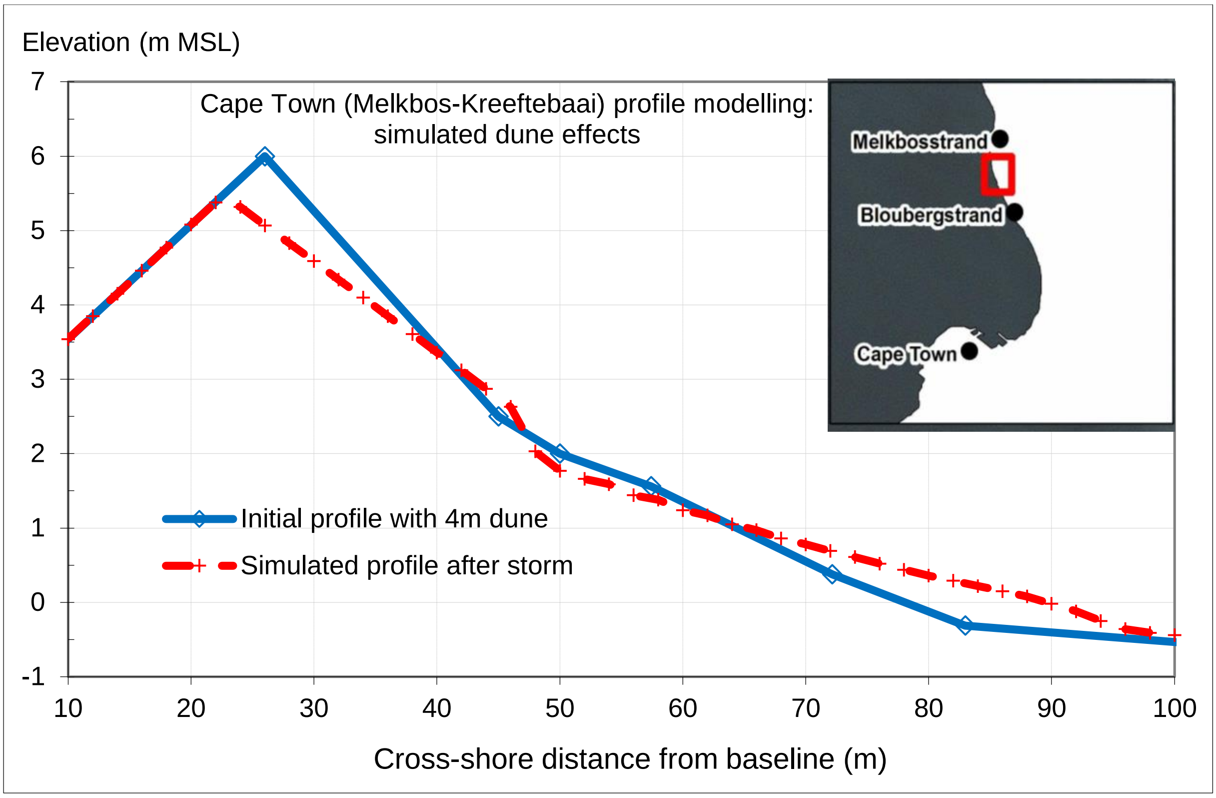


Figure 7.7: Modelling of dune effect at Melkbos-Kreeftebaai site, example with a simulated 4 m high peaked dune.

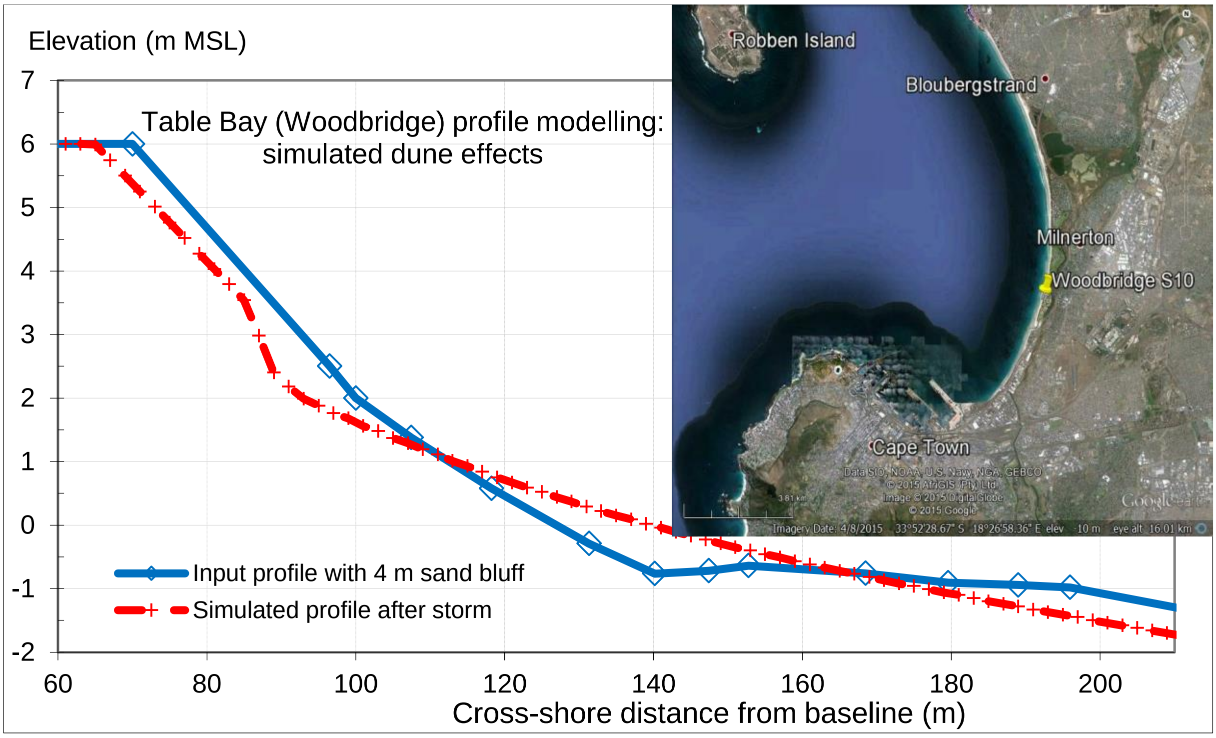


Figure 7.8: Table Bay modelling of dune effect, example with a simulated 4 m high bluff (+2 to +6m MSL).

<u>The main results were as follows:</u>

The same procedure as described before for the Richards Bay and Walvis Bay tests was employed here. The predicted erosion due to the “30 August – 1 September 2008” storm waves at the Melkbos-Kreeftebaai site, for example with a simulated 4 m high peaked dune, is 7 m (at +2 m MSL, Figure 7.7). Similarly, the predicted erosion due to the same storm at the Table Bay site, for example with a simulated 4 m high bluff, is 3 m (at +2 m MSL, Figure 7.8). The predicted erosions for the other input profiles (various dune configurations as listed above) range from 0 to 18 m. Further interpretation of the results is discussed in the following section.

7.3. Dune volume versus setback line distance relationships

Based on the foregoing profile modelling, the relationship between erosion distance and dune volume was examined. The profile volume was calculated as the volume (or area if a unit longshore distance is assumed) above a defined contour up to a fixed distance inland from this contour. In other words, for example, the area above the +2 m CD contour on the upper beach up to 100 m inland from this contour, as indicated in Figure 7.9. The "Volume Ratio" (VR) was then determined by dividing the new profile volume (including a dune: red + blue areas in the example sketch) by the reference profile volume (no dune: red area only in the example sketch). The "Setback Ratio" (SBR) was simply determined by dividing the predicted erosion (at say +2 m MSL) for the new profile (including a dune) by the predicted erosion distance (at the same elevation) for the reference profile (no dune). By plotting these two ratios against each other it was investigated whether a meaningful relationship between these ratios could indeed be established.

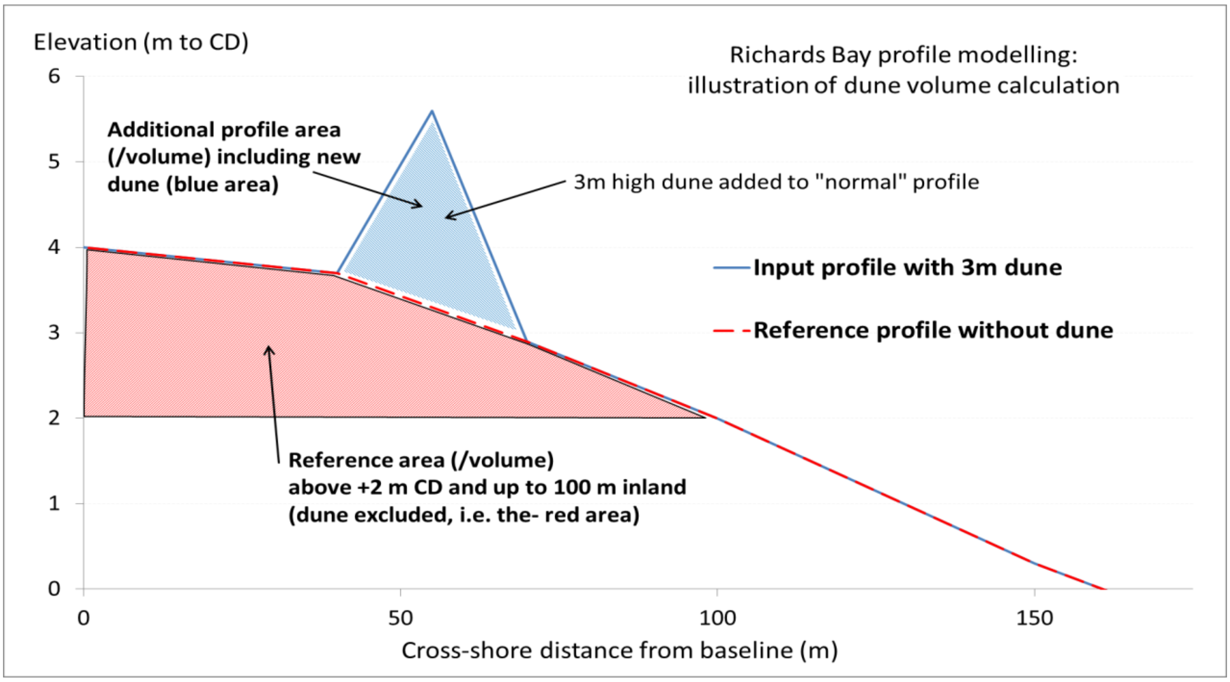


Figure 7.9: Example sketch illustrating how dune volumes are calculated for the "Volume Ratio"

It was found that a functional relationship (where the erosion distance decreases as the dune volume increases) could be established, but that this was sensitive to how the profile volume is calculated. The best relationship was obtained when the profile volume was determined as the volume (or area) above the +2 m MSL contour up to 50 m inland from this contour. There is reasonable justification for this in that the +2 m MSL contour is near the average high water mark. The area between this level and 50 m inland thereof is approximately equivalent to the main area within which shoreline erosions occurs in South Africa, and a dune located within this area would have the most significant effect.

The effect of the dune area (as defined above) on the VR and SBR ratios at each of the three test sites is depicted in Figure 7.10. Also shown are best fit quadratic polynomial equations fitted through the Richards Bay and Walvis Bay series, as well as through all the data of the three sites combined. The best fit lines are all relatively similar and lie within a relatively narrow band as can be seen from the figure.

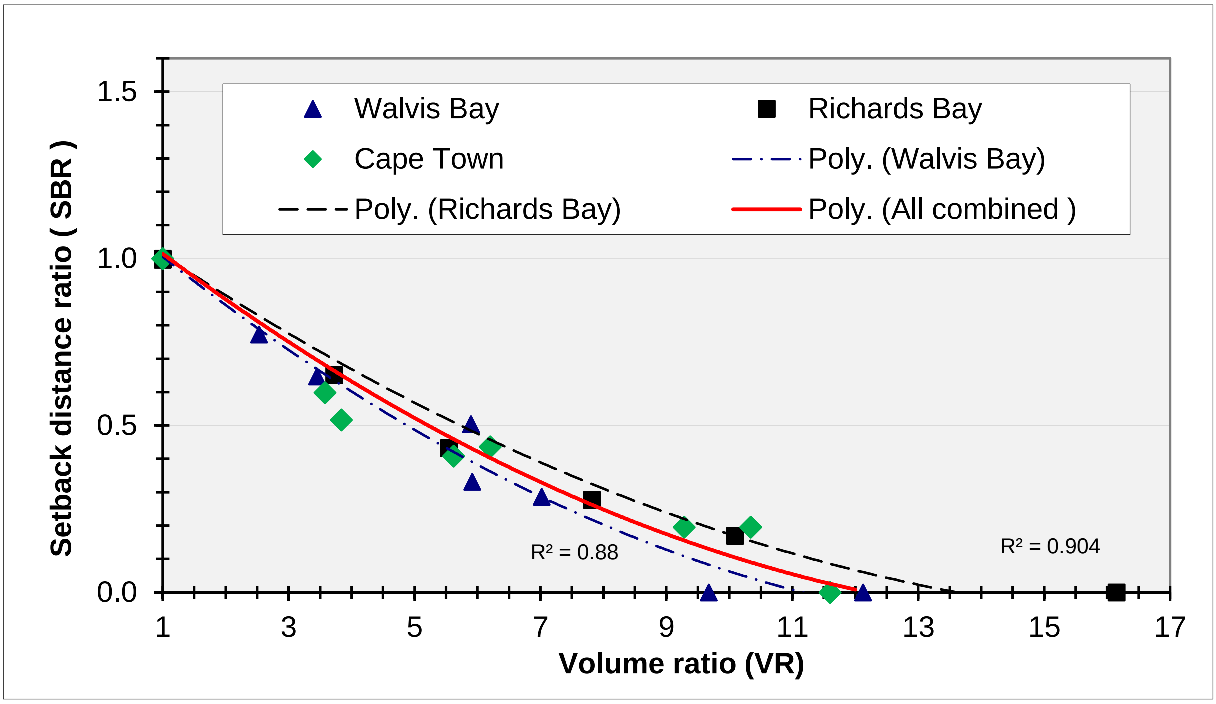


Figure 7.10: Relationship between Dune Volume and Setback Distance

Thus, a rounded off overall relationship was determined, which applies relatively well in all of these cases (Figure 7.10):

$$\mathbf{SBR} = \mathbf{0.005VR^2 - 0.15VR + 1.16} \qquad \textbf{(\textit{Equation 7.1})}$$

If a volume ratio of for example 2 has been found (as calculated from the profiles), then it is found that the setback distance ratio (SBR) = 0.88, as can also be read off from the combined polynomial curve in Figure 7.10. Thus, for this example, the conventionally derived setback distance would be multiplied by a factor of 0.88; in other words, the setback distance would be reduced by 12% in this case. The SBR ratio is therefore the factor by which setback distances determined by other methods should be modified to account for additional erosion protection provided by a dune. Note, that the equation (7.1) should only be applied when the volume ratio (VR) is larger than 1 and smaller than 10. The general application of the “dune methodology” in determining setback lines is described in more detail in Section 7.5.

7.4. Dune Methodology: Applicability and Sensitivity

The sensitivity of the results was also tested by changing the values of some of the main input parameters that are difficult to determine (wave height, wave period, grain size to a lesser degree), to see what effect this would have on the predicted erosion distance. Therefore, considering the typical inaccuracies in recorded data, relatively large variations (of the input parameters) were assessed, namely, the wave height was increased by 25%, the wave period was increased by 32% and the medium grain size was decreased by 14%. All three of these actions are expected to result in significantly increased erosion setback distances. The model results indicated that the combined effect of these actions resulted in an increase of 43% in the erosion setback distance. This shows that the SBEACH model (which was applied to develop the dune method) does respond in the correct manner, and is not overly sensitive to significant changes in the input values. However, it also shows that a conservative approach is warranted when applying such modelling for setback studies. The percentage change selected for each of the above parameters is consistent with the larger inaccuracies that sometimes do occur with field input data, and the combined effect (correctly) resulted in a relatively large difference in the predicted erosion. If, for example, the 1-in-50 year erosion would be estimated directly from profile modelling runs then it becomes critically important to select the correct input conditions (such as the wave conditions, also combined with other factors such as storm duration). If profile modelling is to be used broadly for setback studies, then it is strongly recommended that the sensitivity of the profile model be evaluated extensively.

Dunes sometimes contain layers of material that are more or less susceptible to erosion than the contiguous dune sand (e.g. layers containing cohesive material or hard layers may be found within the dune). The assumption made here in the development of the “dune methodology” is that the dune is homogeneous consisting of only dune sand. This assumption would be true in many areas and is also often assumed in other methods of predicting coastal dune erosion. The assumption of homogeneous dune sand has to be assessed on a site specific basis.

It is postulated that the relationship in Equation 7.1 ought to be applicable for most of the South African coastline. The test locations were specifically selected to be on different seabeds and have different beach characteristics. Thus, the foregoing relationship (Equation 7.1) is expected to be robust and generally applicable within South Africa. Ideally this assumption should, however, be verified further (and it is recommended that this be investigated in a continuation of this research).

7.5. A practical methodology to take into account the effect of dunes

7.5.1 "Dune" setback line methodology for large study areas

Basically there are three possible scenarios that may occur, which relate to the location of the dune on the cross-shore profile. The definition of these scenarios (numbered A, B and C below) and the recommended procedure for each are as follows:

<u>Scenario A</u> – a dune located within the envelope of recorded shoreline locations (Figure 7.11)

If a dune is located within the envelope of recorded shoreline locations, then it has already had a direct effect on the observed shoreline variability. By virtue of its protective properties (buffer sand volume), such a dune would have reduced the shoreline variability. Therefore, its effect on extreme erosion events would already be directly taken into account by most methods or models for predicting erosion that are based or calibrated on the recorded shoreline response.

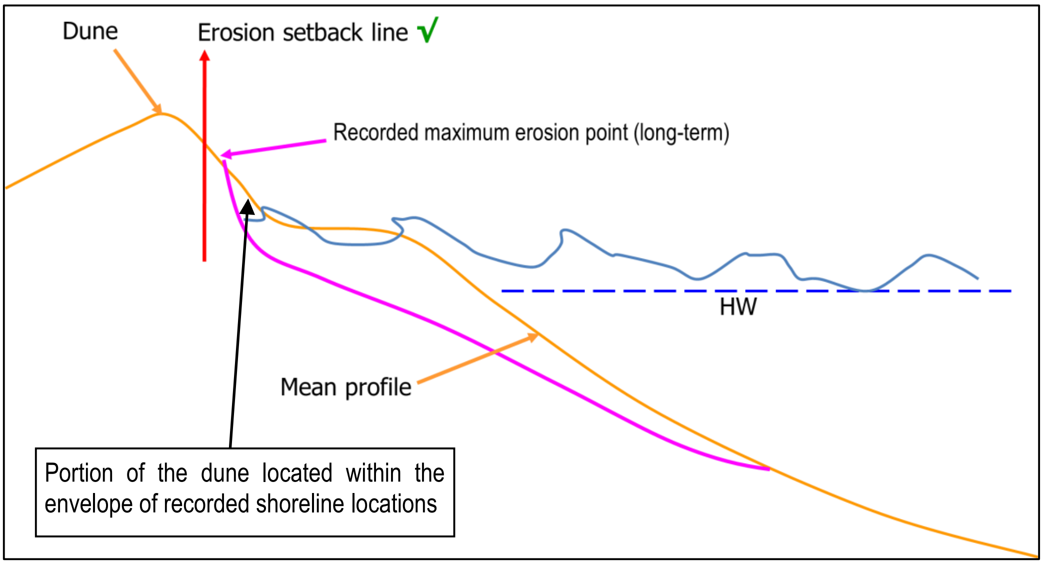


Figure 7.11: Definition sketch for Scenario A – a dune partially located within envelope of recorded shoreline locations

Under such a scenario, the recommended procedure is then quite straightforward in terms of accounting for dune effects. This entails determining the shoreline variability in the usual manner (as discussed in Section 6.2) and calculating or modelling the erosion setback based on the recorded shoreline response. The dune effect is inherent in the recorded data which carries through into the results, and explicit consideration of the effect of dunes is not additionally accounted for in the determination of the erosion setback. Thus, no further adjustment (reduction) of the erosion setback distance is made on account of dunes categorised by Scenario A.

Scenario B – a dune located landward of the predicted erosion setback (Figure 7.12)

The starting point here is to determine the setback distance by means of the usual methods (thus accounting for extreme erosion events, say one in 50 years) and based on the recorded shoreline variations, but not explicitly accounting for dune effects. If a dune is located beyond this distance (Figure 7.12), then it will not significantly affect the erosion that is expected within the planning period (for example the next 50 years). In other words, the dune is located too far landward from the beach to provide significant protection for the area between the beach and the setback line. No further adjustment (reduction) of the erosion setback distance is made on account of such dunes. Although the erosion setback would, therefore, not be reduced under Scenario B, properties or infrastructure located landward of such dunes would nevertheless be subject to additional protection via the dunes (buffer sand volume).

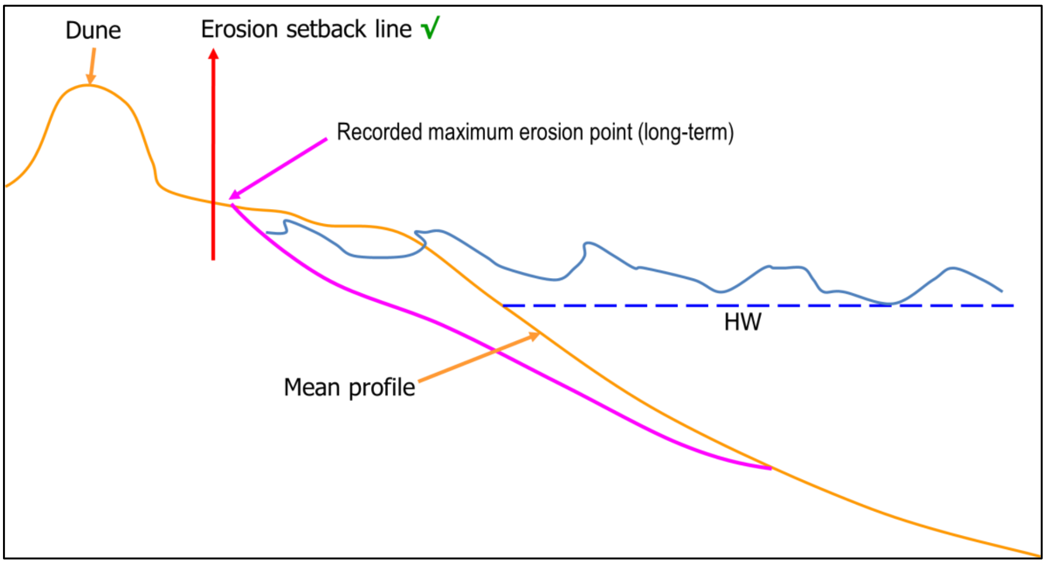


Figure 7.12: Definition sketch for Scenario B – a dune located fully landward of the predicted erosion setback

Scenario C – a dune located landward of recorded shoreline envelope, but within the predicted erosion setback (Figure 7.13)

If a dune is located beyond the envelope of recorded shoreline locations, but still within the usually determined (say one in 50 year) setback distance (i.e. not explicitly accounting for dune effects) then the setback distance should be adjusted according to the relationship given in Equation 7.1. Such a dune would not have affected the recorded shoreline variability, but it will provide additional protection against erosion expected to occur within the planning period (say the next 50 years). To have a noticeable effect (i.e. decreased setback distance) it is estimated that the dune should have a height of more than 1.5 m (from base to crest) and a base width of more than 10 m (minimum volume of 7.5 m^3 per meter alongshore). If the dune is smaller than this no further adjustment (reduction) should be made to the setback distance (which is therefore a slightly conservative approach).

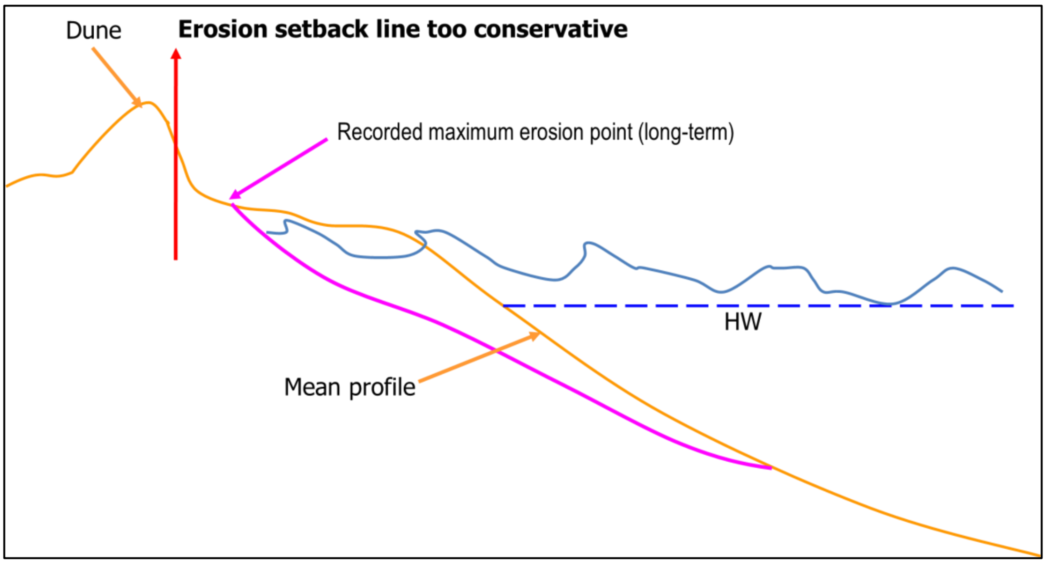


Figure 7.13: Definition sketch for Scenario C – a dune located landward of the recorded shoreline envelope, but within the predicted erosion setback

To determine the final erosion setback distance, the following steps are required:

- Calculate the area of the existing profile (including dune) between the plus 2 m MSL contour to 50 m landward of this contour and above this level (as explained in Section 7.3 and illustrated in Figure 7.9).
- Calculate the profile area of the same profile but excluding the dune. In other words, an estimate has to be made of what the profile would look like without the dune. In many instances this will be straightforward (for example, as illustrated in Figure 7.9), but in some instances this may require sound judgment. What should be kept in mind here is: what part of the feature (for example dune or bluff) would provide additional protection over and above what would "normally" be expected? If an estimated profile is assumed such that the

calculated “extra” area (or volume) provided by the dune is effectively reduced, this implies that a more conservative approach is being taken.

- Calculate the VR ratio. VR = existing profile area (including a dune) / reference profile area (no dune).
- From the relationship given in Equation 7.1 (or from the curve given in Figure 7.10) determine the SBR ratio. Note, that this equation should only be applied when the volume ratio (VR) is larger than 1 and smaller than 10.
- Calculate the final erosion setback distance. Final setback distance = “usual” (say 1-in-50 year) setback distance (as described under Scenario B above) x SBR ratio. For example, if VR = 2 then SBR = 0.88, in other words the final setback would be reduced by about 12% from the “usual” setback distance (where dune effects are not explicitly considered).
- Use experienced judgment to assess whether this is a reasonable final answer.

It is important to note that all of the above scenario methods (i.e. A, B and C) are all solely for the calculation of the erosion setback distance mainly related to extreme events such as storm erosion. Other factors also need to be considered, such as long-term trends, buffer zones, steep slopes, and sea level rise (as discussed elsewhere, e.g. Chapters 2 and 9). Appropriate additional setback distances for each of these factors also need to be added to the erosion setback distance, as discussed further in Chapter 9.

7.5.2 Profile modelling for detailed setback line determination in limited study areas

For detailed setback line studies of smaller areas, SBEACH modelling (or more complex models such as for example XBeach [Roelvink *et al*, 2009]) can be conducted with various profile configurations to assess more directly the effect of these configurations on the erosion distance. The relationship determined before between the volume ratio and the setback ratio (Equation 7.1), can still be used as a good first estimate, but direct modelling can provide more detailed (potentially more accurate) results. Ultimately, a whole range of conditions should be simulated or a number of (different) representative storms be simulated. Results can then be directly analysed statistically or shoreline variation can be estimated from the modelling results.

7.6. Conclusions

In this chapter the effects of dunes on determining setback lines are investigated. It is found that the current methods of determining setback lines have not adequately taken dune effects into account. A widely applied cross-shore morphology model was utilized to compare and calibrate the effect of dunes on setback distances. A generalized dune volume versus setback line distance relationship was determined (Equation 7.1). The input “data requirements” are modest, and the method can be efficiently applied in large study areas. It is suggested that the relationship in Equation 7.1 should be applicable for most of the South African coastline. The test locations were specifically selected from different regions, are on different seabeds, and also have diverse beach characteristics. The method yielded good results for all of the test sites, which well represented the two types of sandy coasts that occur most commonly in South Africa (as characterised in Sections 2.2 and 2.8). Thus, the above relationship is expected to be robust and generally applicable within South Africa. This assumption could be further verified in a continuation of this research. It is contended that, at least, we now have a tentative relationship that can be applied (with the necessary care) to take account of the impact of dunes on setback lines, which to date has been largely neglected.

Specific practical recommendations are provided on how to apply the methodology to take into account the effect of dunes when determining setback lines. This research and the derived relationship (Equation 7.1) can also be well employed to easily show (and quantify) the potential advantage of maintaining, rehabilitating or re-instating dunes as a natural shoreline protection measure against coastal erosion.

Chapter 8: Other components and aspects of setback lines

Chapters 5 to 7 have dealt with the key issues of coastal flooding, shorelines changes and coastal erosion, and dunes. Various other components of, and requirements for, setback lines have been identified in the literature review (Chapter 2), and from the assessment of geophysical coastal hazards and spatial vulnerability (Chapter 4). These other components and requirements are dealt with in this chapter.

8.1 Planning horizons for setback lines

Regarding the coastal processes setback line, in South Africa the acceptable risk was traditionally defined to be that the storm erosion and flooding line may be exceeded once in 50 years, while a sufficiently wide vegetated buffer still remains (e.g. Theron, 2003a, 2003b). For major infrastructure developments, or structures with design life-spans exceeding 50 years, the 1-in-100 year setback line was sometimes selected as more appropriate (e.g. Breetzke *et al* 2008). More recently in South African practise, DEADP (2010), made strong argument for applying a 100 year planning horizon in determining setback lines. Van Weele *et al* (2014) provided recommendations related to considering the nature of development activities proposed in the coastal zone in relation to the planning horizon applied in decision making. “Decisions regarding land-use and development can then be based on either the value of the proposed development or activity or the nature of the proposed activity.” This proposal was originally stated in DEADP, 2010 (Table 8.1).

Internationally, countries such as the USA, the Caribbean, Spain, Romania, and Australia apply similar planning horizons (from 1-in-30 to 1-in-100 year, e.g. Fenster, 2006; WAPC, 2003; NCCOE, 2004), while New Zealand takes a more conservative 1-in-150 year flooding level into account in terms of coastal processes and governance planning (New Zealand Local Government Guidance Manual, 2014). In European countries, the design of sea dykes generally aims at a failure probability regarding overtopping with a return period in the order of greater than 1-in-100 year to 1-in-200 year, while in the Netherlands, the design criterion for the central part of the coast is a return period as high as 1-in-10 000 years (Kron, 2013). According to Li *et al* (2014), a dune failure probability equivalent to 1-in-100 000 years was chosen for the most vulnerable parts of the Netherlands.

Table 8.1: Planning horizons for setback lines based on value and risk of failure of infrastructure (adapted from Van Weele *et al*, 2014; originally from DEADP, 2010):

Value of Infrastructure	**Life of Infrastructure**	**Impacts of Failure of the Infrastructure**
Low **(up to R2 million)** Recreational facilities, car parks, board walks, temporary beach facilities	Short-term Less than 20 years	Low Minor inconvenience, alternative facilities in close proximity, short rebuild times
Medium **(R2 million to R20 million)** Tidal pools, piers, recreational facilities, sewerage pump stations	Short- to Medium-Term Between 20 and 50 years	Medium Local impacts, loss of infrastructure and property
High **(R20 million to R200 million)** Beachfronts, small craft harbours, residential homes, sewerage treatment works	Medium- to Long-Term Between 50 and 100 years	High Regional impacts, loss of significant infrastructure and property
Very High **(Greater than R200 million)** Ports, desalination plants, nuclear power stations	Long-term In excess of 100 years	Very High Major disruption to the regional and national economy, failure of key national infrastructure

Looking at the European practise, it may be argued that the determination of setback lines at a time scale of even 100 years is not adequate. It should however be kept in mind that the South African context is very different. While the low-lying Netherlands are obviously potentially subject to great risk regarding human life from flooding, this is not the case in South Africa. In comparison, the South African coastal relief is much steeper with no low-lying inland areas, and only relatively narrow coastal strips where development or infrastructure is at risk from the sea in limited areas. In fact, in recent setback line determinations in South Africa, there has been very strong public opposition (seemingly mainly from an economic perspective) to applying planning horizons exceeding 1-in-50 years (e.g. Breetzke *et al*, 2012 and Van Weele *et al*, 2013). On the other hand, it may be argued that in South Africa even residential development is unlikely to be relocated after 50 years and that at many locations of high economic value the determination of setback lines at a time scale of 100 years may not be adequate.

Thus, it would seem that the most appropriate planning horizon to consider in determining setback lines in South Africa would be a minimum of 50 years for typical coastal facilities, and 100 years for residential properties and important coastal infrastructure (e.g. small craft harbours), with 100 years in fact preferably for any structure expected to remain in service for longer than 50 years. Critical or strategic infrastructure, such as major ports or nuclear power plants, need to consider even longer time horizons (200 to 1000 years). In considering appropriate planning horizons, it should be kept in mind that, statistically speaking, within a period of 50 years, there is a 63% probability of the 1 in 50 year storm occurring. In keeping with the precautionary principle, the goal of long-term sustainable development (both in synergy with the South African ICM Act), and uncertain expectations of progressive climate change effects, *the argument is put forward here* that it may be prudent *to consider a more conservative planning horizon of 100 years for all coastal setback lines in South Africa.* This may potentially hinder some "short-term" mainly developmental (i.e. financial) gains, but will ultimately be more beneficial from a "triple bottom line" or long-term sustainability (including public financial) perspective. From the legal perspective, it is the responsibility of the relevant government authorities to make the final decision on which planning horizon to apply in determining a coastal setback line.

8.2 Accounting for climate change effects/impacts in setback lines.

8.2.1 Climate Change effects, projections and future scenarios relevant to setback lines

It is now widely accepted by many scientists that global warming is (most probably) affecting the metocean climate, which will potentially have various physical impacts on the coast (e.g. Allsop, 2005; Houghton, 2005; IPCC 2001, 2007, 2013; Mimura and Kawaguchi, 1996; Theron 1994, 2007, 2011; etc.). Feagin *et al* (2005) go further in stating that it is now also widely accepted that erosion rates are accelerating as a result of sea level rise induced by atmospheric warming. Therefore, coastal development and governance also needs to consider, and include as far as possible in planning, potential coastal climate change (CC) impacts. The expected impacts of CC in the coastal zone have been broadly discussed in Sections 2.5 and 2.6, but aspects more specifically related to setback lines are discussed below and in the following sections

Changes in the configuration and profile shape of sandy coastlines depend on a number of factors of which the most important are the metocean and hydrodynamics drivers, and the availability and distribution of sediment. Sand along the coast is moved mostly by waves, while the waves approaching the coast are in turn affected by bottom topography (Theron *et al*, 2010a). As the sea level rises, existing topographic features will be located in deeper water and will have a different effect on waves approaching the coast. Features landward of the breaker zone will be in deeper water and will either have an amplified or dampened effect on the wave climate compared to the present.

Also, higher sea levels will require smaller storm events to overtop existing storm protection measures (Theron, 1994); see Figure 8.1 for an example of an existing problem area. Deeper water features may deepen to the degree that their effect on the inshore wave climate is negligible. The points of wave energy convergence and divergence will change. The new locations of wave energy convergence could be expected to experience an increase in erosion while those locations currently subject to energy convergence could accrete if they are exposed to less energy in future (Ewing and Michaels, 1991). Changes in wave approach will change longshore currents and longshore sediment transport. An example of the repercussions of cascading effects is that, while there may be a slow increase in global sea levels, coastal areas presently protected by low-lying reefs may become more exposed much sooner due to the combined effects of increased water depth (SLR) and more extreme wave-climate (Theron, 2007).

Figure 8.1: An existing overtopping and flooding problem (Table Bay, South Africa), likely to worsen due to climate change *(Photo: L van der Merwe)*

According to Mather *et al* (2009), current SLR rates for the South African east coast are estimated to be about + 2.74 mm/yr-1, which is within the global range of values found elsewhere. Based on a review of the post 2007 literature and findings, it was concluded in Section 5.2.4, that the most appropriate (or ‘central estimate’) of SLR by 2100 is ~ 0.85 m to 1 m, with a plausible worst-case scenario of 2 m (including “accelerated” SLR) and a low estimate of 0.5 m. The corresponding best estimate (mid-scenario) projections for 2030 and 2050 are about 0.15 m and 0.35 m, respectively. It is further concluded that the insufficient quality and quantity of the existing South African sea-level data as well as the paucity and uncertainties of regional projections for Southern Africa do not yet provide a sufficient basis for applying different regional projections of *eustatic* SLR along the South African coast. At present, the future scenarios most appropriate for determining setback lines and longer term planning for the South African coast, appear to be to apply the consensus of global *eustatic* SLR projections (as summarised above). Certainly there is not sufficient hard evidence or confidence in current projections to assume lower (less "safe") projections for parts of the South African coast.

8.2.2 Future metocean climate and wave climate scenarios

A review of the available information (Section 2.7) indicates that despite some possible trends in certain parts of the world, potential changes in oceanic wave regimes resulting from global warming are still very uncertain in all parts of the globe. In the most recent IPCC report (IPCC AR5 SPM 2013), figures (images) are provided which indicate possible future changes in wave height over the Southern Ocean. These could potentially be interpreted to yield very rough indications of possible deep-sea wave height increases off South Africa, but with insufficient resolution and very low confidence. A 5% increase in storm surge by 2060 off Cape Town as applied in Luger's recent study (Luger, 2012), is not sufficiently well founded for general application. Such a small increase in one of the smaller components of coastal flooding elevations would in any case have a very small effect when determining setback lines. In the available literature there are still no clear future predictions or scenarios available providing details on future wind/wave climate off the South African coast and it does not appear to be sensible to speculate further about this until better information becomes available. In view of the paucity of quantitative information, a variety of plausible future wave scenarios is not (yet) forthcoming, with which to enable a wide assessment of the potential impacts of changes in regional weather systems and oceanic wind fields. Based on the literature review and discussion in Section 2.7, it is concluded that currently an appropriate scenario for future wave climate off the South African coast should be a 6% to 10% increase in wave height by 2100. The potential effects of such scenarios for future wave climate off the South African coast of a 6% and 10% increase in wave height by 2100 have been included in the wave runup case studies provided in Section 5.3. The potential climate change effects of a possible wave height increase of 6% or 10% by 2100 on coastal erosion can similarly be estimated by means of the Parametric model (Equation 6.6).

Having found no publications whatsoever linking potential climate change effect on currents and tides to South African coastal processes and sediment dynamics and therefore lacking any clear direction, other potential marine CC effects (e.g. warmer sea temperature, acidification, Agulhas current changes, rainfall and sediment yield changes, etc.) have not been considered further in terms of the setback line issues.

8.2.3 Cross-shore shoreline response to increased seawater levels (SLR)

Hard erosion resistant shores

When assessing the erosion potential of the coastline due to SLR, coastal areas first need to be characterised in terms of two main shoreline geomorphological characteristics, i.e. hard erosion resistant shores, or sandy erodible beaches. Hard, erosion resistant shores are usually rocky or have been “artificially” armoured, e.g. revetments, seawalls, breakwaters, etc.

Hard erosion resistant shores will generally respond to sea level rise as follows: in most instances, there will be no noticeable erosion as such (and for example the Bruun Rule, which is discussed later in this section, is not applicable). However, the high-water line will still move landward according to the slope above the present high-water line. For example, if the present slope is 1 in 10, then a rise in sea level of 0.5 m or 1 m means a landward movement of the high-water line of 5 m or 10 m respectively. Slopes above the high-water line are usually much steeper than those of the sub-aerial beach or nearshore profile, especially those of rocky shorelines. Therefore, in most instances, the landward movement of the high-water line along hard rocky shorelines is only expected be in the order of a few metres. However, in a few unusual situations, where the slope above the high-water line is very flat, the landward movement could be in the order of 10s of metres. For example, if the slope is 1 in 50, then a rise of 0.5 m or 1 m means a landward movement of 25 m or 50 m respectively.

Soft rock shores

“Soft rock” shores are composed of rocky material that is only partially resistant to erosion. Such shorelines are more prone to wave erosion than hard rock shores, but still usually much more erosion resistant than sandy shores. Over periods of decades to a century small but noticeable erosion is often observed, and in lower wave energy environments, discernable amounts of erosion products remain visible on the shoreline for some time.

Walkden and Dickson (2008) developed a process-based mathematical model of rocky shoreline recession, the SCAPE model. This time step model includes the rock strength, sea-level rise, wave height and period, cliff height, sand content, proportion of rock comprising sediment suitable for building a beach, tidal range, beach volume and hydrodynamic constants. The model iterates towards a dynamic equilibrium profile and stabilization of the long-term average. Walkden and Dickson report that once dynamic equilibrium was reached, recession rates were found to be well represented by a basic relationship across all parameter values tested. Their relationship for projected shoreline recession E_2 (applicable to a composite soft rock low volume beach) is as follows:

$$E_2 = E_1.(S_2/S_1)^{0.5}$$

Where E_1 is the historical shoreline recession,

S_2 is the projected sea-level rise, and

S_1 is the historical sea-level rise.

A limitation of this relationship is that it is only valid over extended time scales; thus projections of less than 100 years may not be feasible. The biggest practical problem for general application in South Africa, is that for much of our coast, data on historical shoreline recession does unfortunately not exist and observed recession (from historic aerial photography) is mostly insignificant or very small. Never-the-less, where such data does exist, this technique may well be applied and could yield good results being based on the actual local shoreline response to historical sea-level rise. However, the veracity of this method has to date not yet been demonstrated in South Africa.

The cross-shore shoreline response of sandy erodible beaches to increased water level (SLR) is discussed in the following section.

Sandy erodible beaches

In South Africa, the potential impacts of sea level rise (in terms of shoreline erosion) have been taken into account in most setback line studies since 1991 (e.g. CSIR, 1991). The most widely known (and applied) formula for determining recession (erosion) as a result of sea level rise was proposed by Bruun (1988). The so-called "Bruun Rule" is applicable to sandy beaches and does not take account of cliffs or rocky coastlines. The Bruun rule can be applied to give a first estimate of possible erosion of 'soft' sandy beaches. In some cases, broad dunes and wide beaches could mitigate such erosion to some degree. In other situations narrower beaches backed by hardened dunes will resist erosion resulting in less erosion than predicted by the Bruun rule. In fact, there are some types of coastal conditions where application of the Bruun rule is inappropriate (Theron, 1994; Cooper and Pilkey, 2004). Hands (1977) found that in areas having broad active profiles or low back shore, offshore or longshore sediments sinks, as well as in areas where the eroding backshore contains a large percentage of material that would be unstable as a nearshore deposit, the ratio of retreat to submergence would be even larger than estimated by means of the Bruun rule. Narrow active profiles, higher back shore sediment deposits, coarse grain sediments and increased supplies of sediment from outside the area considered will, on the other hand, all tend to diminish the ratio of shore retreat to submergence (Theron, 2007). The shoreline response is therefore also affected by the resilience afforded by certain natural features and processes, such as dunes and abundant/depleted sediment sources. The Bruun formulation ("rule") for the recession (R) due to SLR is given below:

$$R = \frac{S \times L}{H_d + H_b}$$

Where: S is the sea level rise in metres

L is the distance to the profile closure depth

H_d is the profile closure depth

H_b is the height of the beach berm

Thus, the main parameters that are taken into account in Bruun's unsophisticated rule are the amount of sea level rise (S) and the slope of the nearshore ($L/(H_d + H_b)$), as schematically illustrated in Figure 8.2.

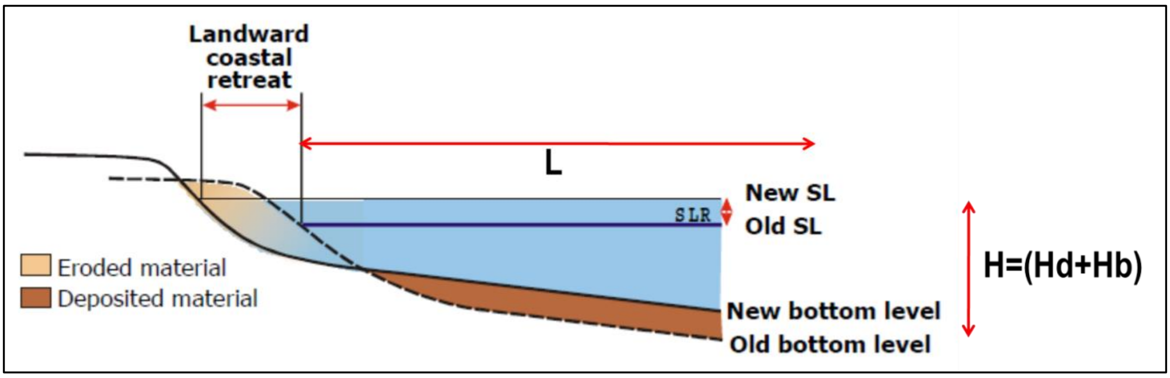


Figure 8.2: Schematic illustration of the Bruun model of profile response to rise in sea level showing erosion of the upper beach and nearshore deposition. *(Adapted from Davidson-Arnott, 2003)*

The profile closure depth can be determined from the nearshore wave data and the sediment characteristics in this area. (The profile closure depth is defined as that depth below the water surface at which no significant changes in the profile occur.) The theoretical methods of Swart (1974), Hallermeier (1981) and Birkemeier (1985), can be used to calculate this depth. In rare incidences where sufficient profile data is available, the closure depth can be assessed directly from the observed sub-aerial profile changes. Typical closure depths in very exposed South African locations are in the order of 15 m (to perhaps 20 m in the most extreme cases), and progressively less than 15 m as the location becomes less exposed (more sheltered from direct wave impact). If the bottom slope from around 0 m MSL to about -15 m MSL (or even -20 m MSL) is relatively uniform, then the exact calculation of the closure depth is not important, because the value for $L/(H_d + H_b)$ will remain constant, as determined by the slope.

Thus, in the above manner, the potential impacts of sea level rise can be investigated using Bruun's rudimentary erosion rule, to determine the additional setback required for SLR. The Bruun rule is also useful as an indicator of where more detailed investigation of future impacts may be required in case of detailed engineering design studies for specific developments. Mather and Stretch (2012) tested the applicability of the Bruun rule for the KZN coast by using a series of 14 paleoshorelines as the recorded data, and found that the Bruun rule was able to predict the observed recession to within 10% of the actual retreat. This good result provides significant confidence in the application of the Bruun rule in South Africa.

Application of the Bruun rule – Table Bay case study

Based on Bruun's erosion rule and SANHO bathymetric charts of Table Bay, the potential shoreline erosion for three selected scenarios were investigated, namely SLR of 0.5, 1.0 and 2.0 m (as per Section 5.2.4). (The nearshore slopes ranged from 0.018 to 0.004.) The Bruun model predicts that the areas along the Table Bay coastline with a steeper nearshore slope will erode between about 100 m to 370 m for the given scenarios, while the areas with relatively milder or flatter nearshore slopes are predicted to erode between about 20 m to 80 m. The ranges and average potential erosion for each sea level rise scenario are presented in Table 8.2. It should be noted that in many cases, where hard structures or large dunes are found, that these may significantly reduce or virtually halt such horizontal shoreline erosion (assuming that they are maintained or remain intact).

Table 8.2: Potential erosion according to Bruun's rule in the Table Bay coastline area.

Amount of sea level rise (m)	Potential erosion range From (m)	To (m)	Potential erosion Average (m)
0.5	21	94	51
1	41	187	103
2	83	374	206

The implications for the northern Table Bay coastline of the possible combined effects of shoreline erosion due to SLR and high wave runup were also investigated. For example, for a 1-in-20 year wave height, a sea level rise of only 0.5 m would lead to a wave runup 4.6 m to 6 m above present MSL (based on Nielsen and Hanslow's (1991) model as described before in Section 5.3.4). In addition, according to Bruun's model, the 0.5 m sea level rise could result in average erosion of 50 m. This means that the already high wave runup point (elevation of 4.6 m to 6 m to MSL) would in addition shift landward by about 50 m. Thus, the combined longer term impacts of higher storm wave runup levels and potential coastal erosion due to even higher SLR scenarios (1 m to 2 m) are anticipated to have major consequences. Figure 8.3 illustrates what the combined impacts of shoreline erosion and higher wave runup could mean for a 0.5 m rise in sea level and a 1-in-20-year sea storm.

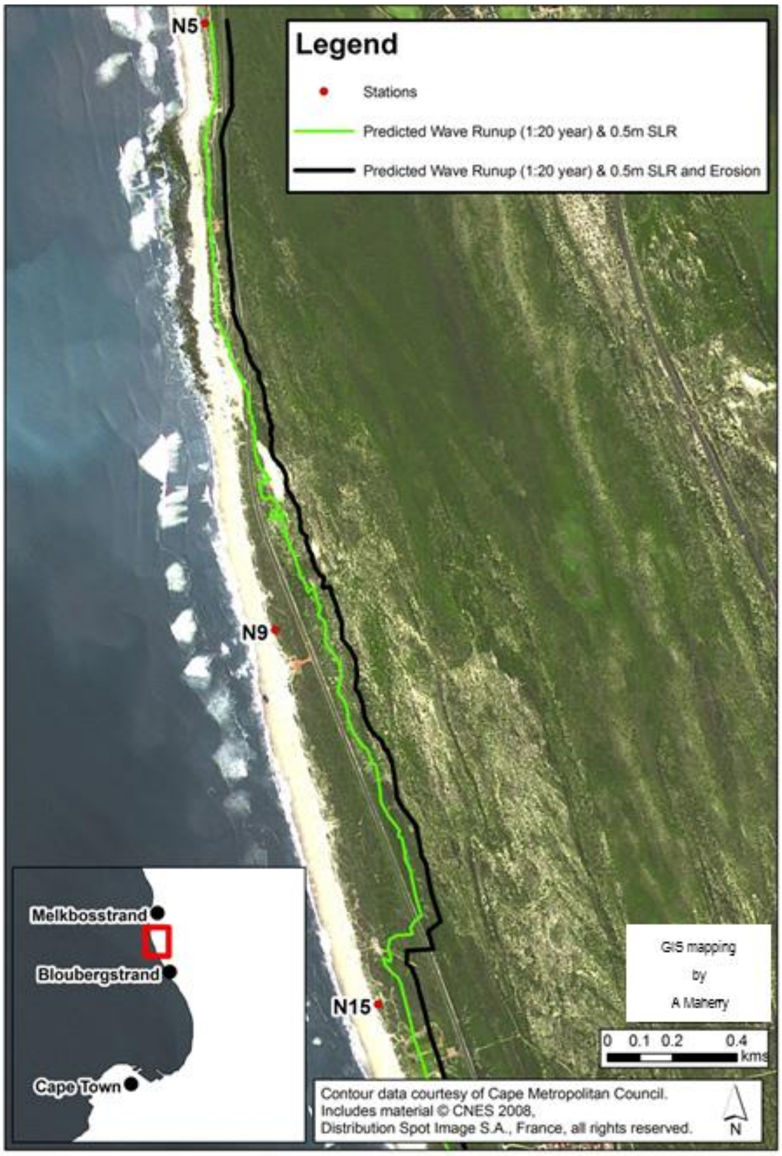


Figure 8.3: Predicted combined effects of potential shoreline erosion with Bruun's rule and higher wave runup for 0.5m rise in sea level and a 1-in-20 year sea storm on northern Table Bay coast.

Alternative methods to quantify cross-shore effects of SLR on sandy shores

An alternative approach to the Bruun rule that is sometimes applied, assumes a rudimentary upward transferal of the existing profile (similar to that described for the rocky slopes before), in response to a rise in sea level. For sea level rise of 0.5 m and a slope of 1 in 30 this would give a recession distance of 15 m, for example. This "slope transferal" approach generally yields much lower setback distances than the Bruun rule. Although the Bruun rule is very simplistic and not always applicable, it is based on both limited field observations and a logical explanation of physical coastal processes, which gives it some credibility. The "slope transferal" method (typically yielding low setback values), on the other hand, is generally considered to yield results that are not sufficiently conservative for typical South African beaches, but does give an indication of the *minimum* additional setback required to cater for SLR effects.

Stive (2004) and Davidson-Arnott (2003), present some of the few alternative methods to Bruun's Rule, which are potentially better, but more demanding ways of assessing shoreline response to SLR. Du Bois (1977) and Dean and Maurmeyer (1983) modified the Bruun rule to derive the "Transgressive Barrier" model, but this model is mainly applicable to barrier island coasts, which do not occur in South Africa. In another modification, Dean and Maurmeyer (1983) modified the 2D Bruun rule into a 3D model by incorporating longshore sediment transport terms, to take account of gradients in the longshore drift. This is, however, only useful where such gradients do occur (often unknown in South Africa); when there is no gradient the equation reduces back to the 2D Bruun rule. Such gradients can in any case be easily accounted for separately and do not actually form part of the shoreline response to SLR. Thus, these two modifications of the Bruun rule do not really contribute to practical improvement of the Bruun rule (for South African application), and it is perhaps not surprising that they seem to have been rarely applied at all.

One alternative method, the so-called "Rollover model" (Carter, 1988), has also been applied in a few instances in South African studies (Cooper, 1995a, 1995b). According to this model, the shoreline migration rate (dR/dt) is equal to the rate of sea-level rise (ds/dt) over the "basement slope" ($\tan_B$). The "basement slope" is equivalent to the bottom slope from the shoreline (MSL) to the profile closure depth. Assuming for example, a sea-level rise of 0.35 m over the next 50 years and a basement slope of 20/700, this model gives a shoreline migration rate of 0.25 m/a. The Rollover model thus predicts recession of 12.3 m in this case over the next 50 years. Under certain conditions, the Rollover model predicts erosion of similar magnitude to Bruun's rule. The Rollover model may be more applicable to barrier beaches, which are not common in South Africa (they do occur at some South African estuary mouths).

Bray and Hooke (2007) advocate a technique which basically amounts to extrapolation of a historical shoreline evolution trend analysis. Their expression for projected shoreline recession R_2 is as follows:

$$R_2 \quad = \quad R_1.S_2/S_1$$

Where R_1 is the historical shoreline recession,

S_2 is the projected sea-level rise, and

S_1 is the historical sea-level rise.

This technique would obviously be applicable to areas where such data exists, and is expected to yield good results being directly based on the actual local shoreline response to historical sea-level rise. For much of the South African coast such data unfortunately does not exist and observed recession (from historic aerial photography) is often very small, making the application of this technique problematic.

Mixed sandy/rocky shorelines

Sand veneer beaches underlain by rocky substrate

It is known that some South African shorelines consist of mainly sand and cemented coastal dune sand (e.g. Figure 8.4, along the KZN coast). Thus, intermittent rocky outcrops (i.e. cemented coastal dune sand) are observed along such shorelines (e.g. Figures 8.4 and 8.5), while several other dune and more sandy beach areas are typically underlain by such rock at various depths below the sand surface level. Due to the partially rocky nature of the shoreline, the potential coastal erosion could in these areas be less severe than for a wholly “soft” (i.e. consisting of sand alone) shoreline. The potential erosion would be reduced, because of the presence of the rocky outcrops which effectively “pin” the shoreline in place, where these outcrops occur.

Figure 8.4: KZN example of a shoreline consisting of sand veneer beaches underlain by rocky substrate. *(Photo A Theron)*

The rocky substrate, seen along some parts of the coast, would thus tend to reduce erosion of the beach in these areas. Under circumstances of background erosional forces due to (amongst others) progressive SLR, the finer sediments will tend to be preferentially lost even though large scale erosion may initially be less obvious. The underlying rocky substrate areas, which currently have crest levels above intertidal elevations, and are located within the beach profile or are underlying the dune, would probably become more emergent and may become progressively more so in the future, due to the ongoing deficit of finer sediments. Rocks that are currently emergent within or seaward of the intertidal zone, would become progressively inundated due to SLR in the future.

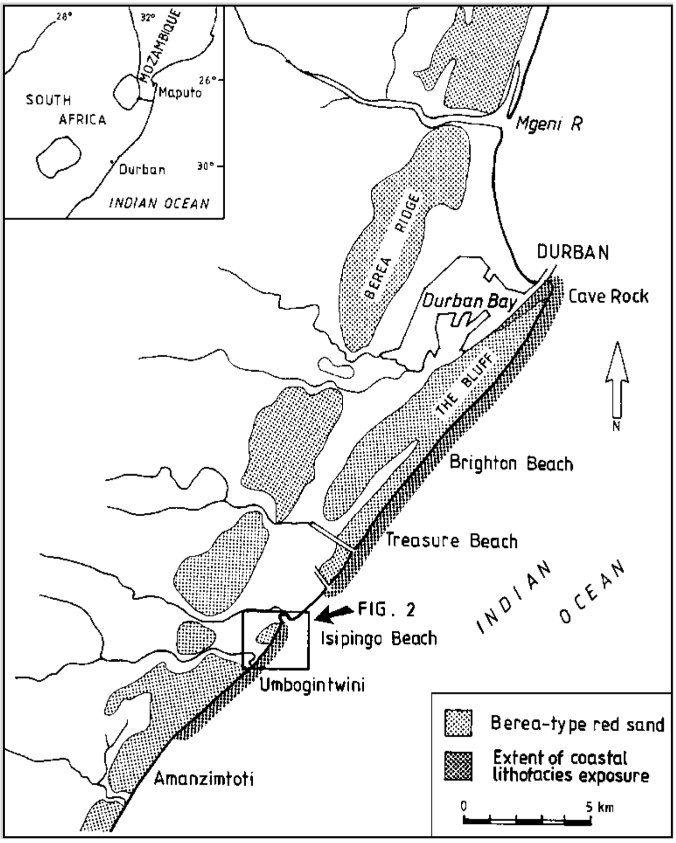


Figure 8.5: Locations of cemented sand outcrops south of Durban *(From Cooper and Flores, 1991).*

It should be pointed out, that in some instances, intermittent rocky outcrops can in fact result in even greater localised erosion "hot-spots" (in the order of up to 100 m) in *adjacent* areas during storm events (e.g. Figure 8.6), or when such areas are starved of sediment (Phelp *et al*, 2009; Smith *et al*, 2010). However, this is not related to climate change effects, but due to (temporary) interruptions of the longshore sediment supply during times of high cross-shore losses (e.g. due to sea storms or surf zone currents).

Figure 8.6: Beach erosion at Amanzimtoti before (a) and after the storms in 2007 (b). *(From Phelp et al 2009).*

<u>Sandy shorelines containing significant amounts of pebbles and/or cobbles</u>

Gravels (pebbles and cobbles) found on some South African shores (often around the high-tide line), such as for example found along parts of the Algoa Bay shoreline, will also tend to dampen potential erosional effects. Similar to the situation discussed above relating to rocky outcrops, under circumstances of background erosional forces due to (amongst others) progressive SLR, the finer sediments will again tend to be preferentially lost even though large scale erosion may initially be less obvious. The pebble and cobble layers within the beach profile would probably become more emergent and may become progressively more so in the future, due to the ongoing deficit of finer sediments.

<u>Estimating recession of mixed sandy/rocky shorelines due to sea-level rise</u>

In case of such “mixed sandy/rocky” shorelines types as discussed above, a first estimate of the *maximum possible* potential recession due to SLR can still be made by means of the Bruun rule in many instances. A less conservative result may be derived by appropriately reducing the depth of closure when applying the Bruun rule. This assumes that the inshore profile steepens towards land, as is very often the case. (The logic applied here is similar to that of Komar *et al*, 1991, who found that overestimates of the depth of closure will result in longer cross-shore profiles, and consequently produce larger values of shoreline recession when applying the Bruun rule.)

Alternatively, the “slope transferal” method (typically yielding low setback values), may be applied to give an indication of the *minimum* recession expected due to SLR effects (as discussed above under the alternatives to the Bruun rule). The most appropriate result for the "mixed sandy/rocky" shorelines types discussed above, would then be between this minimum value and the maximum value according to the Bruun rule applied with full closure depth.

Sandy shores located landward of extensive surf zone reefs.

Coastal areas characterised by sandy shores located landward of extensive surf zone reefs should be treated with caution when assessing potential cross-shore effects of SLR. The methods discussed above to assess recession due to SLR are not applicable here. Storm waves approaching the coast are affected by bottom topography, and shallow reefs that cause wave breaking dissipate much of the incident wave energy. However, as the sea level rises, existing topographic features including such shallow reefs will be located in deeper water and will have a reduced effect on waves approaching the coast. Areas landward of the reef breaker zone will experience an amplified wave climate compared to the present. In other words, these reefs currently provide protection from wave attack to the inshore areas and beach sands that are susceptible to erosion. If the coast is subjected to the predicted sea-level rise, the protective role of the reefs will be diminished. An example of such a potential situation is the Gordon's Bay – Strand shoreline, as depicted in Figure 8.7.

Figure 8.7: Extensive reefs sheltering the Gordon's Bay – Strand shoreline from high wave energy *(Photo H Theron)*

Such special situations should be thoroughly studied by determining the expected future wave conditions at the shoreline through proper wave modelling, or by considering depth limited wave conditions for the projected future seawater levels. Methods such as those described in Section 6.2 can then be employed to assess the potential erosion resulting from the SLR and amplified wave conditions.

8.3 Vegetation / wind-blown sand buffer areas

Infrastructure developments susceptible to wind-blown sand problems in coastal beach/dune systems include parking areas, ablution and recreational facilities; housing development for temporary or permanent occupation (e.g. Figure 8.8) and storm-water drains (adapted from Rust and Illenberger, 1996). Besides the important role that dunes can play in reducing coastal erosion (Chapter 7), another very important function of vegetated dunes is to prevent or reduce wind-blown sand problems (Figure 8.9).

Figure 8.8: A house located within a natural sand pathway and inundated by wind-blown sand *(Photo courtesy of DEA)*

Wind-blown sand (or aeolian) transport rates may be estimated by various theoretical methods (for example, Horikawa *et al* 1986, Pye and Tsoar, 1990, and Swart, 1986). The theoretical methods are mostly based on formulae derived for dry, non-cohesive sand of unlimited quantity blowing over a flat, unvegetated surface under constant wind conditions. Since these criteria are seldom met in practice, the calculated transport rates can be considered to be potential rates only, with true rates usually differing significantly. The main parameters affecting the aeolian transport rate are the wind regime, the wind fetch, and the distance over which the sediment can be mobilized (beach area), the sediment characteristics, and other factors such as sand moistness, sediment armouring, and vegetation cover. Some previous setback line studies have included calculations of predicted aeolian transport rates (Chapter 2). Although such largely theoretical transport rates can be informative, in fact no direct link between these rates and recommended vegetated buffer zone widths is found in any of the literature reviewed in Chapter 2.

Of more importance is to actually consider the characteristics of the wind regime within study areas. Based also on the range of sediment characteristics found on South African beaches and the grain sizes yielding the highest aeolian transport rates, areas prone to significant wind-blown sand are subject to wind velocities above 12 km/h, that dominate during the dry season with an onshore component more than about 20% of the time. In addition, the observed pressures from human activities (to vegetation cover), as well as the size and orientation of actual existing dry beach areas (the source areas) need to be assessed (which can be done in situ or by remote means). The real requirements for vegetated buffer zone widths in South Africa have been well established by trial-and-error through ongoing coastal zone management activities by local authorities (e.g. Figure 8.9) and experience of dune rehabilitation and maintenance practitioners (albeit virtually unpublished in formal literature). Thus, it has been determined that along the South African coast, the minimum practical width that is necessary to effectively trap wind-blown sediment, ranges from about 10 m to 40 m, depending on circumstances (Tinley 1985, Heineken and Badenhorst, 1999; Theron 2008). The low end of this range (10 m to 20 m) is applicable to the typically coarser grained, narrower and steeper beaches found along KZN, further characterized by wetter conditions and prolific vegetation growth (even in the absence of irrigation) with the rainy season coinciding with the hotter summer months. The top end of the range (30 m to 40 m) is applicable to areas such as the Cape, with hot and windy dry summers where rehabilitated and pioneer dune vegetation are slow growing and struggle to establish without irrigation.

Figure 8.9: A vegetated buffer area at Table View alleviates wind-blown sand problems and reduces risk to landward development

Thus, theoretical predictions of aeolian transport rates as previously calculated in some South African setback studies were of little practical use, and the actual requirements applicable to South African conditions have been well established by trial-and-error. The recommended vegetated buffer zone widths provided here can be directly applied in setback studies, and if required, the expert knowledge and experience of specialists (e.g. botanists) can be sought in major dune management, rehabilitation or maintenance projects. If a more site specific determination is required, then the computation of the wind-blown sand transport can be useful to objectively classify the transport rates as being say, low, medium or high. Such determination includes accounting for the directions of the main wind-blown transports, which also impact the width of vegetation that is required to trap this sand. In conjunction with the relevant specialists (e.g. botanists) the vegetated buffer zone requirements can then be specified.

It is sometimes possible to incorporate (part of) the required wind-blown sand buffer area within the area seaward of the erosion line, if it is accepted that this buffer area will be partially washed away from time to time (as occurs naturally on many beaches). Limited erosion “damage” can in many instances be repaired through natural vegetation growth, but in some instances or where more substantial storm damage has occurred, active human intervention is usually required to rehabilitate the area. Good guidance to assess and address such issues as well as the significant problems associated with dune blow-outs, is provided in Barwell (2011), and Mclachlan and Burns (1991).

Breetzke *et al* made specific setback provision for “sediment pathways” by identifying potentially mobile dune fields from historic aerial photographs and advancing the line landward to the most inland position of the dunes. A general recommendation of locating setback lines at the landward border of dune fields is a logical step to avoid wind-blown sand problems and conversely to avoid damage to such environments from development. This is basically in agreement with various other authors (e.g. Tinley, 1985; Burns *et al*, 1993). However, such “strict” measures may be argued by some to be overly conservative, and have given rise to legal challenges where development opportunities of existing land ownership may be impacted. Nevertheless, strictly in terms of “safeguarding” infrastructure and amenities and “sustainable development” such (arguably) conservative recommendations are preferred and are in keeping with the intent of the ICM Act. A general rule is not appropriate here and each such case (of which there are fortunately limited instances in South Africa) should be assessed on its own merits in conjunction with all relevant stakeholders. However, such partially socio-economic considerations are beyond the scope of this thesis.

8.4 Instability of steep sandy slopes, coastal bluffs and cliffs

8.4.1 Steep sandy slopes and dunes

The state of the primary or frontal dunes within a study area can also dictate a cautionary approach in determining the setback line (e.g., Figure 8.10). The steep seaward slope of the primary dune comprises a very fragile environment.

Figure 8.10: Illustration of the steep seaward slope of a primary or frontal dune *(Photo A Theron)*

In some instances, the erosion setback line, as determined by the usual means, would be located on the steep seaward slopes of coastal dunes or bluffs. In practice this could mean that fixed structures in these areas would be built on too steep slopes. (The gradients of 1:6 under stable conditions and 1:10 under unstable conditions can be used as definitions of a steep slope.) Any disturbance of the vegetation or other human activities on the slope could destabilise the dune slope (which may cause slumping of the steep dune face) and may also cause additional wind-blown sand problems. This would be undesirable as it would lead to degradation of the environment and costly maintenance of developments. Thus, it is strongly recommended that any fixed structures should be very carefully sited and constructed with due care of the environment (preferably landward of the dune), or located where the slope is sufficiently flat.

Areas where the dune slope is very steep and where the space between the high-water line and the dune foot is very narrow are particularly high-risk areas for development and, as such, no development should be allowed here. In these areas, the development setback line needs to be located further inland and landward of the "edge" of the dune (from where the dune slopes down steeply seaward). Geotechnical aspects such as foundation stability should also be considered in planning any construction relatively close to the crest of or on the slopes of the primary dune.

For dunes of relatively low height (<10 m from base to crest), in assessing dune stability, it is usually sufficient to simply consider the angle of repose as the *maximum* potential stable slope angle. (Mather, 2012, also basically applied slip failure angles to account for stability of steep slopes.) However, in accordance with the guideline provided above, a *conservative* acceptable slope angle may be as mild as of 1:6 under stable conditions and even as gentle 1:10 under unstable conditions. For dunes of more than 10 m in height it is advisable to involve geotechnical engineers and to conduct a slip circle analyses, especially if the slope is steeper than 1:6. Where dune heights exceed 30 m to 40 m the involvement of geotechnical engineers becomes an imperative for any proposed development near such dunes. Alternatively a conservative setback from the dune crest of at least two to three times the dune height is required.

A case in point is the Northern KZN coast near Richards Bay. This shoreline is backed by high dunes which are subject to large episodic dune slips. A previous geotechnical assessment of this area has indicated that the phreatic surface and the emergence of seepage water along the shoreline influence slope stability (DLP, 2008). Analyses of aerial photography reveal that dune slips in this area are associated with an average dune retreat of about 60 m, while a few very large slips resulted in dune retreat of about 100 m, with maximum alongshore extent of up to about 500 m. In this case, a minimum additional setback provision of about 110 m is warranted to safeguard again dune slip impacts. (Typical dune heights here are about 35 m to 40 m, which means the 110 m provision is in the order of three times the dune height.)

8.4.2 Erosion setback provisions for coastal bluffs, cliffs and rocky shores

Geomorphological processes and predictions

<u>Cliffed coasts</u>, consisting of hard (weather and wave resistant) rock, will tend to be virtually static or erode by amounts hardly noticeable over decadal time scales. Cliffed coasts consisting of softer material (prone to weather and wave erosion), are often already undergoing a slow long-term erosional trend (e.g. Figure 8.11). Although the high-water line would tend to respond to SLR in the same manner as for hard cliffs, sea level rise and especially increased storminess, may increase the

rate of cliff retreat. A possible local example, is Swartklip (Northern False Bay coast), which has been reported as subject to low rates of long-term erosion (Schoonees and Bartels, 1991).

***Figure 8.11: Example of a cliffed coast near Stilbaai undergoing slow long-term erosion** (Photo A Theron)*

Coastal bluffs are synonymous with coastal cliffs, but are usually composed of weakly or moderately cemented (lithified) sands or sediments, or contain some cohesive sediments which enable the typically steep bluff profile to form. Coastal bluffs can be subject to continuous erosion and in some cases, episodic landslide failures or large slip circle type failures. According to Collins and Sitar (2008), the main bluff failure modes are due to: (wave) undercutting or over steepening of the profile of the bluff, by rotational ("slip circle") failure, by tensile fracture caused by stress relief or loss of soil strength, or from lateral or vertical inertial forces from seismic shaking. Specifically, Collins and Sitar (2008) found that bluffs composed of weakly cemented sands (unconfined compressive strength—UCS between 5 and 30 kPa) fail principally due to oversteepening by wave action with maximum slope inclinations on the order of 65^0 at incipient failure; while bluffs composed of moderately cemented sands (UCS up to 400 kPa) principally fail due to precipitation-induced groundwater seepage, which leads to tensile strength reduction and fracture. These findings reveal that in order to accurately quantify the minimum required setback provision for steep dune/bluff/cliff collapse, may require specialist inputs based on geotechnical investigations.

Kamphuis (1987) developed a useful expression for bluff erosion (R) based on the wave power per unit length of shoreline, with a correlation (r^2) of 0.81 to field data. His expression is:

$$R_{mean} \quad = \quad 1.06(P_{mean})^{1.37}$$

Where R_{mean} is the long-term average recession rate of the shoreline in meters per year, and P_{mean} is the long-term average wave power in kilowatts per meter arriving at the shoreline. Although this expression was developed for (cohesive) glacial till bluffs which is, therefore, different to typical South African coastal bluffs, it may well be suitable for application in South Africa in a modified form. However, the fact that the determination of the wave power requires the wave parameters at the breaker line, makes it unsuitable for easy application along large study areas.

Guidelines for setback provisions

Council for the Environment (1986, 1991) defined a cliff as a steep rock or soil face that usually faces the sea and has a height from toe to crown of more than 10 m. They provide the following guidelines pertinent to setback line provisions:

- No development should occur at the toe, on the face or immediately behind the crown of the slope or cliff;
- A minimum setback should be enforced to make provision for a buffer strip of natural or stabilizing vegetation behind the crown of the slope or cliff;
- The width of the buffer zone should be determined by qualified experts. This width will vary depending on natural characteristics (e.g. vegetation cover, soil type, rainfall, water table) and the scale of the proposed development.

Pertinent guidelines provided in the literature for cliffs, bluffs and rocky shores can be summarized as follows:

- Cambers (1997): cliffs (limestone and volcanic) - 15 m from the edge of the cliff; low rocky shores - 30 m from the natural coastal vegetation line;
- Polish regulations make provision for a setback along cliffed shores of 100 m landward from the cliff edge (Celliers, 2010);
- Breetzke *et al* (2012) applied a “storm retreat line” for rocky shores along the top of steep slopes;
- USA States: (MI) bluffs are high risk areas – setback provision for 30-year erosion protection plus 5 m buffer; (PA) bluff setback of 50 times the annual rate of recession from the bluff

face for residential, and 75 times the annual recession rate for commercial properties (Celliers, 2010).

The range of recommended setback provisions, contained in these guidelines for each of the coastal types, are thus as follows:

- Cliffs - 15 m to 100 m landward from the edge of the cliff;
- High rocky shores along the top of steep slopes; low rocky shores - 30 m from the natural coastal vegetation line;
- Bluffs – 30 times the annual recession rate plus a 5 m buffer, up to 75 times the annual recession rate.

A practical means of obtaining a first estimate of the erosion potential of coastal bluffs, cliffs and rocky shores, is to assess the geologic and geomorphologic characteristics of the study area. The pertinent coastal geologic characteristics can be arranged in order of most erosion resistant to most erosion prone, as follows: hard rocks (Magmatic), "medium" hardness rocks (Metamorphic), soft rocks (Sedimentary), non-consolidated coarse sediment, non-consolidated fine sediments. The coastal geomorphologic characteristics can similarly be arranged in order of least to most erosion prone, as follows: mountains, rocky cliffs, erosive cliffs, sheltered beaches, exposed beaches, tidal (or other) flats (which usually consist of finer sediments, sometimes including some silt or clay fractions); dunes, and river mouths. In South Africa, tidal (or other) flats are mainly found within estuaries, due to our generally high energy coast.

Thus, magmatic and metamorphic rocks, mountains and rocky cliffs are generally not subject to significant coastal erosion within typical setback line planning horizons (20 to 100 years). The general *minimum additional* erosion setback provision for such hard erosion resistant "stable" shorelines in South Africa is here recommended as follows:

- Cliffs - 15 m landward from the edge of the cliff;
- High rocky shores - 6 m landward from the top (crest) of steep slopes (this 6 m provision also makes allowance for public access as discussed further in Section 8.6.2);
- Low rocky shores - 30 m landward from the natural coastal vegetation line (established woody shrubs or trees). (In extreme situations this should be checked against coastal flooding simulations to ensure adequate setback provision has been made.)

Soft rocks (sedimentary) and erosive cliffs may be subject to significant coastal erosion within typical setback line planning horizons. The best practical means of assessing erosion setback requirements for such shorelines, is to analyze historic recession of these shorelines by means of, for example, aerial photography spanning at least 30 to 50 years, but ideally going as far back as possible (which is typically in the 1930s or 1940s). In special cases or complex situations, the assistance of geotechnical experts or geotechnical engineers may be required to provide more detailed assessment and recommendations. The general *minimum additional* erosion setback provision for such shorelines in South Africa is here recommended as 30 times the annual recession rate plus a 5 m buffer. However, if the setback planning horizon is say 50 or 100 years, then the *additional* erosion setback provision should be set at the respective period (i.e. 50 or 100 years) times the annual recession rate. Even if the historic recession rate of such coasts is estimated to be close to zero, *additional* erosion setback provision needs to be provided. Similar to the recommendations made for steep dunes, for soft rocks and erosive cliffs or bluffs of more than 10 m in height, it is advisable to involve geotechnical engineers and to conduct a slip circle analyses, especially if the slope is steeper than 1:6. Where the cliff or bluff heights exceed 30 m to 40 m the involvement of geotechnical engineers becomes an imperative for any proposed development near such features. For preliminary planning purposes a setback from the cliff or bluff crest of at least two to three times the height is required.

Shorelines characterized by non-consolidated coarse or fine sediment, beaches and dunes, are prone to significant or pronounced erosion and should be investigated in depth with the erosion setback requirements being determined according to the methods and procedures provided in Chapters 6 and 7.

8.5 Estuaries and river mouths

8.5.1 Estuary/river mouth dynamics

River mouths are highly dynamic areas subject to large change (e.g. Keurbooms Mouth shoreline configurations depicted in Figure 8.12). Spits or sand bars and channel embankments at river mouths are highly dynamic areas and inherently unsuitable for development. Although the position of the mouths may appear to be fairly stable, past experience, as well as assessment of aerial photographs, indicate large variability in many instances around the South African coast. In general, development on areas adjacent to river mouths that are part of the littoral zone is not recommended. Thus, setback lines determined in the usual manner along shorelines facing the sea are not applicable at river mouths and are discontinued in these areas.

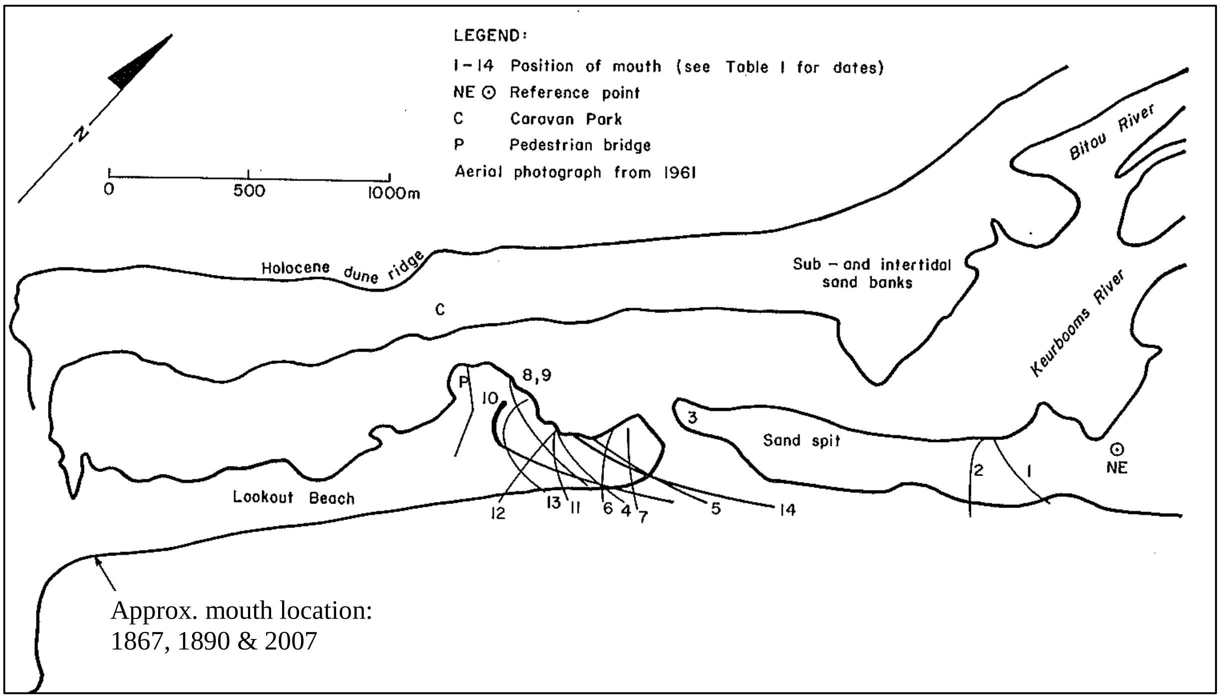


Figure 8.12: Keurbooms Mouth shoreline changes (1867 to 2007) *(adapted from Duvenhage and Morant 1984)*

Other examples of large mouth meandering of shifts in location include the Eerste River Mouth (Figure 8.13), and the Mlalazi Mouth (Figure 8.14). The Eerste River Mouth shifted by a few kilometres alongshore (from right to left in Figure 8.13), in the process under-scouring a pump-house and endangering facilities at Macassar.

Figure 8.13: Large shift in location of the Eerste River Mouth *(Photo A Theron)*

The Mlalazi Mouth (located south of Richards Bay in KZN) seems to be migrating in a northerly direction (Figure 8.14).

Figure 8.14: Northward shift of the Mlalazi Mouth *(Photo A Theron)*

A practical and robust means of assessing mouth dynamics and possible channel meandering, is to analyze historic shoreline changes in the vicinity of river mouths by means of, for example, aerial photography spanning at least 50 years, but ideally going as far back as possible (which is typically in the 1930s or 1940s). Note, that in the example of the Keurbooms Mouth (Figure 8.12), the mouth has only been located at the far southern end of Lookout Beach (far left in Figure 8.12) three times since 1867. This shows that all available information, covering as long a period as possible, should be considered. In special cases, the assistance of geophysical estuarine specialists or engineers specializing in river/estuarine hydraulics/morphology may be required to provide more detailed assessment and recommendations. Erosion setback requirements in such locations are therefore strongly informed by historic data and information, and need to be conservative, as the maximum extent of progressive changes or long-term shoreline evolution is difficult to foresee or predict. DEADP (2010) state that if mouth migration tendencies occur, provision for mouth channel meandering must be allowed for, considering historical trends, geological constraints and beach and dune topography.

8.5.2 Estuarine reaches inland of the mouth

In South Africa three basic options are currently employed for delineating “setback lines” or “buffer areas” along estuarine reaches inland of the mouth (Van Niekerk, 2011):

i. In-depth detailed numerical hydrodynamic (or river hydraulics) modelling studies taking into account a wide range of factors (e.g. topographical surveys, backwater curve analyses, surface roughness coefficients and vegetation cover, super critical flows), to derive the 50 or 100 year estuarine floodlines. Some such studies are less reliable through not properly considering mouth state/dynamics (i.e. mouth open/closed or dynamic mouth dimensions regarding depth and width), or seawater levels in conjunction with river floods.

ii. Pre-selected elevation contours e.g. +5 m or +8 m MSL contour (e.g. 5 m contour provided by Chief Directorate Surveys and Mapping, or DEMs (Digital Elevation Models) extracted from LiDAR data). As an example, the demarcation of the +5 m MSL contour around the Groot Brak Estuary is depicted in Figure 8.15.

iii. Lateral boundaries delineated based on “associated wetlands, intertidal mud and sand flats, beaches and foreshore environments that are affected by riverine or tidal flood events” (Van Niekerk and Turpie 2012, p.31).

Figure 8.15: The demarcation of the +5 m MSL contour around the Groot Brak Estuary. *(GIS mapping by A Maherry)*

These are the same options as identified in the City of Cape Town setback line study (CoCPT 2012a: 41-45). The assessment and determination of “setback lines” or “buffer areas” along estuarine reaches inland of the mouth, is a whole study area in itself, and is not focused on in this thesis. Depending on the detailed level of quantification required in comprehensive studies of individual estuaries, the expertise of geophysical estuarine specialists or engineers specializing in river/estuarine hydraulics/morphology should be employed to provide the necessary detailed assessment and robust “setback lines/buffer areas” recommendations. For current wide scale application along estuaries (excluding KZN), the +5 m MSL contour (plus an additional buffer in certain circumstances), is usually taken as an adequate “surrogate” setback line (until more accurate and potentially more appropriate flood-line modelling may be conducted). The best guidance should soon be forthcoming from the report “Managing South Africa’s estuaries for change, Version 1”, which is presently being drafted under the auspices of Department of Environmental Affairs (Van Niekerk *et al*, 2014).

8.6 Ecological and social components of coastal setback lines

The preceding chapters and foregoing sections of this chapter mainly deal with the geophysical coastal-marine processes, dynamics and components of coastal setback lines, which are indeed the focus of this thesis. For convenience these components will collectively be referred to by the abbreviated term “coastal processes setback”. However, coastal development setback lines (or coastal “management” lines) also have to consider other important aspects. These can be grouped under the main headings of ecological and social components of coastal setback lines. By nature these components include some more amorphous aspects (which may be considered to have more “fuzzy” characteristics) and typically require much wider consultation and public participation in the assessment and demarcation process. (According to Celliers *et al* 2009, successful integrated coastal management (ICM), which includes the promulgation of setback lines, “is often characterised by extensive public consultation and democratic decision-making, a concept that is also entrenched in the Constitution of South Africa, a theme which also runs throughout the ICM Act”.) Although these aspects of coastal setback lines are not the focus of this thesis, they are briefly discussed in the following sections, because they ultimately form part of holistic setback line demarcation. Before a coastal development setback line can indeed be promulgated, all the geophysical and biophysical components, as well as the socio-economic aspects have to be considered.

8.6.1 Ecological components of coastal setback lines

This component of setback lines refers to ecologic, biodiversity, and environmental conservation aspects, and related ICM requirements.

Ongoing coastal development and provision of infrastructure and amenities contribute largely to reductions and losses of ecologically important zones in coastal areas. Thus increased setback provisions are required to compensate or mitigate these ecological impacts.

Anthropogenic activities in the coastal zone also have an indirect impact on the biophysical system through the process of “coastal squeeze”. This is illustrated in the sketch below (Figure 8.16). When shorelines are naturally eroding or receding as a result of ongoing reduction in sediment supply or through sea-level rise, the mean beach profile (or the profile envelope) is usually approximately maintained in the long-term, but moves landward. This means that the area (or cross-shore width) of the different beach zones (intertidal, swash and upper beach) are also largely maintained. These areas also constitute important functional ecological zones. When the backshore area has become fixed and the shoreline is prevented from migrating further landward, the ongoing erosive processes then lead to a narrowing and steepening of the mean beach profile (Figure 8.16). If coastal structures such as seawalls or revetments are then added to protect landward buildings, this can lead to total loss of the beach area (as in the example depicted in Figure 8.17). (Regarding existing high value coastal infrastructure deemed worthy of protection, there may in some instances be very few other viable options.) The steepening and narrowing of the mean beach profile (and its possible total loss) mean that the important ecological zones and habitat areas are likewise reduced or “squeezed” in the process (and ultimately possibly totally lost). Thus, an additional setback provision is required to allow for the landward migration of the important ecological zones and to mitigate the impacts of coastal “squeeze”.

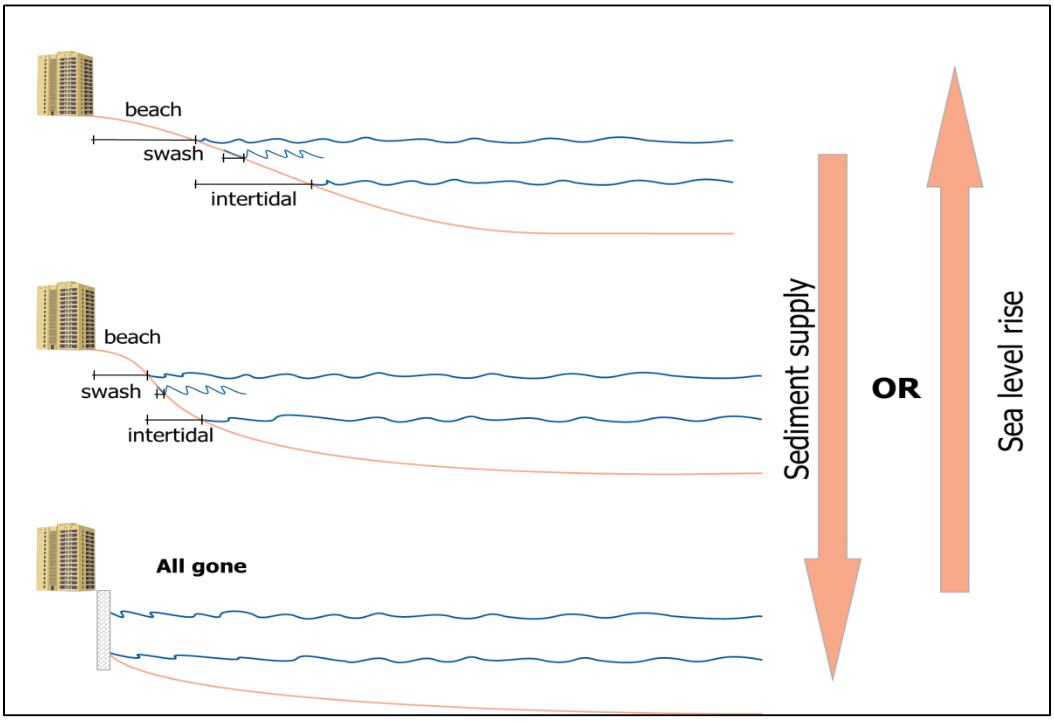


Figure 8.16: Illustration of how anthropogenic activities lead to reductions/losses of ecologically important zones through "coastal squeeze".

The process illustrated in Figure 8.16 is matched by that described in Feagin *et al* (2005): "Beach erosion in a sediment-rich environment results in a natural landward movement of coastal communities so long as there are no obstructions to restrict such movement. Where obstructions, or barriers, do exist and prevent this landward movement of plant and animal communities, erosion reduces the area available to them. Such barriers are usually man made developments such as houses, roads, etc." To address this issue, the Nelson Mandela Bay Municipality has included a requirement in their setback line policy, to "determine environment buffers required inland from the high-water mark to maintain a functional coastal ecosystem under future sea level rise scenarios" (Nelson Mandela Bay Municipality, 2012). Such buffer zone widths will depend on factors such as the recession rate, the importance and functional area requirements of the ecological zones, and social aspects (which are discussed in the following section).

Figure 8.17: Illustration of how a sandy beach habitat may be lost through construction of a revetment to protect infrastructure located too near the sea. *(Photo A Theron)*

Provisions for ecological components of coastal setback lines should be informed by an environmental assessment based on the SANBI biophysical sensitivity layers. Critical Biodiversity Areas can be identified from biodiversity maps provided by nature conservation bodies (e.g. Cape Nature) and SANBI, which include information on the reason for the biodiversity status of the area and proposed management of the area. Such assessments should be conducted by (or in full consultation with) biodiversity specialists or ecologists specializing in the coastal domain. Related coastal areas that should obviously be included (or properly considered) in setback line provisions, are nature and marine reserves, protected areas and conservancies.

Finally, it should be kept in mind that vegetated dunes, mangroves, corals, wetlands (and to a limited degree kelp forests) all have some coastal erosion and/or flooding protection potential and can mitigate coastal climate change impacts to some degree. The objective (actually an important opportunity), therefore, lies in protecting and managing these natural defenses, or indeed in enhancing or expanding their positive effects by increasing such areas where practical or reintroducing such natural systems where they have been lost or impacted. This win-win (or "no regrets") approach serves both man and nature in enhancing both geophysical coastal protection and serving environmental needs, which ultimately enables sustainable coastal development and tourism and long-term social benefits.

8.6.2 Social components of coastal setback lines

Legal and zoning aspects

Implementation of the ICM Act (2008) has made it a legal obligation to determine coastal setback lines in all the South African coastal provinces. Negative public response can be generated by the potential legal implications that may arise as a consequence of the position of a setback line in relation to private properties with development rights. According to the ICM Act the state can "prohibit or restrict the building, erection, alteration, or extension of structures that are wholly or partly seaward of this line", while the Registrar of Deeds must also endorse the coastal setback line in all relevant deeds. Thus, socio-economic "pressures" are brought to bear and especially legal challenges typically arise once (proposed) coastal development setback lines are brought to public notice. Before a setback line can be promulgated, additional legal and "planning requirements" have to be considered, such as cadastral boundaries, private property rights, zoning boundaries, coastal public property, coastal protection zones, Municipal or Town planning zones, military/other special use areas, special management areas, etc. Together with other ICM considerations, public access, heritage, shading, aesthetics, and such considerations, these should all feed into the determination of holistic coastal development setback lines. Extensive public consultation and democratic stakeholder engagement is a crucial part of the process. Some guidance on these issues is provided in Celliers *et al* (2009). However, as mentioned before, these legal, zoning and other social aspects are beyond the scope of this thesis.

Public access

The ICM Act requires that access to the coast and related infrastructure and amenities must be planned and managed to protect coastal resources, their values and public safety. Coastal access as defined in DEA (2014) can be a road, an informal pathway or public parking area, or any number of combinations of these, providing direct access to the sea shore, which can be indicated on a map. "Public coastal access can be referred to as people's ability to view, reach and move along the shoreline. It is therefore a management issue that deals with questions about whether the public can physically use or view the coast, whether the public can legally pass over land to reach the coast, and whether the public can use coastal areas without placing undue stress on ecosystems." DEA (2014). An important aspect related to setback lines concerns making provision for public access ways to the coast over public, state and privately owned land. A practical starting point would be for at least all state entities, and local and regional authorities, to seriously consider what provisions they could practically make to provide such public access ways to the coast, which is part of their mandate under the ICM Act.

Spanish regulations make provision for public coastal access (“easement of passage”) stipulating a 6 m shoreline passage (i.e. parallel to the shoreline) to be permanently clear for pedestrians, as well as an “easement of free public access to sea” stipulating (“cross-shore”) access to the coast at alongshore intervals of 500 m for vehicles and 200 m for pedestrians (adapted from Celliers, 2010). Figure 8.18 shows an example of public access facilities provided for disabled persons in Spain.

Figure 8.18: Example of public access facilities provided for disabled persons in Spain *(Photo A Theron)*

The Spanish regulations appear to provide a good general ideal for urban settings, but would be very difficult to implement in some South African coastal areas, due to extensive private and public/state holdings. At least the 6 m shoreline passage (parallel to the shoreline) should in most instances be practical to incorporate into the setback line provisions. Figure 8.19 shows a South African example of where good pedestrian access has been provided within a vegetated buffer area that is part of a managed dune area. Where setback lines are located relatively far inland from the sea along sandy shores, this 6 m shore-parallel pedestrian access should mostly be available within the setback required for the coastal processes, on condition that dunes, coastal vegetation and public access are all properly managed or preserved and maintained in an integrated manner. If the coastal processes setback line is located close to the high-water mark or if the backshore area is very steep or consists of a cliff or bluff, providing clear shore-parallel pedestrian access at all times is difficult. In such instances additional provision should be made for pedestrian access where practical. Local circumstances, topography, land ownership and other such site specific issues will have to be considered in a pragmatic way to make provision for reasonable public access.

Figure 8.19: Examples of alongshore and cross-shore public access provided at Stilbaai *(Photo A Theron)*

The best guidance should soon be forthcoming from the "Guide for the Designation and Management of Coastal Access in South Africa", which is presently being drafted under the auspices of Department of Environmental Affairs (DEA, 2014).

Aesthetic features

Significant views (or landscapes), some of which define sense of place, such as for example, the R44 route from Gordon's Bay to Pringle Bay (Figure 8.20., or Chapman's Peak drive, etc.) are of socio-economic importance and need to be preserved. Similarly, some aesthetic features (for example unique rock formations) which have significant "socio-economic" value are located within the coastal zone. Thus setback provisions are required to ensure that such "features" are not detrimentally impacted. In practice, a site inspection accompanied by a relevant specialist (possibly a landscape architect/town planner from a local or regional authority) is recommended in order to determine possible buffer areas that may be required around such features.

Figure 8.20: Aesthetic landscape view along the R44 at Kogelbaai worthy of preservation

Heritage

Heritage relates to areas/features or structures of historical heritage or cultural/symbolic importance, that are located within the coastal zone, and which therefore need to be preserved by incorporation into setback lines provisions. Historical structures typical of a certain era or style (e.g. Figure 8.21), or national monuments are some examples of heritage structures to be preserved by making appropriate provision in the demarcation of setback lines. Appropriate provincial authorities (such as Heritage Western Cape) and other such bodies can provide reports and maps indicating heritage sites located within the coastal zone. In practice, a site inspection accompanied by a relevant specialist is recommended in order to determine the buffer area that may be required around such features.

Figure 8.21: Historical fish traps near Stilbaai are of cultural heritage importance *(Photo A Theron)*

Shade

Tall buildings near the shoreline, for example at Strand (Figure 8.22) and Durban (Figure 8.23), can cause shade problems, requiring an extra setback provision. An important attraction of beaches to both local users and tourists is to enjoy the sunshine on the beach. In South Africa, tall structures built adjacent to the beach on the eastern, northern or western side thereof, may cast large shadow areas on the beach, which in some instances can even persist into the late morning or already become evident from the early afternoon, thus significantly detracting from the value of the beach experience. The Ethekwini Municipality has for example employed building restrictions limiting the height of structures near the shoreline to reduce such impacts. The alternative is to locate such buildings further landward (where practical) to reduce the expanse and duration of shadows that they may cast on the beach.

Figure 8.22: Tall buildings near the shoreline casting shadow on the beach at Strand *(Photo A Theron)*

Figure 8.23: Tall buildings near the shoreline casting shadow on the beach at Durban *(Photo A Theron)*

8.6.3 Combining coastal processes setback lines and coastal zone management issues

In practice, the coastal development setback line will always be landward of the coastal flooding and erosion setback line to allow space for wind-blown sand effects and other buffer zone properties. A final crucial further step is to combine the coastal processes setback line with ecological and social aspects and general coastal zone management (CZM) principles that may call for an even more landward shift of the setback line or scrapping development altogether within large distances (hundreds or thousands of meters) from the coast in some sensitive areas.

8.7 Conclusions

Previous chapters have dealt with the key issues of coastal flooding, shorelines changes and coastal erosion, and dunes. All the other components of, and requirements for, setback lines (as identified in the literature review, and in the assessment of geophysical coastal hazards and spatial vulnerability), are dealt with in this Chapter. The pertinent findings (appropriate methods or recommendations) regarding these other components and requirements are provided here under the relevant sub-headings:

Accounting for climate change effects/impacts in setback lines.

The best estimate (or 'central estimate') of SLR by 2100 is ~ 0.85 m to 1 m, with a plausible worst-case scenario of 2 m and a low estimate of 0.5 m. The corresponding best estimate (mid-scenario) projections for 2030 and 2050 are about 0.15 m and 0.35 m, respectively. Currently an appropriate scenario for future wave climate off the South African coast is a 6% to 10% increase in wave height by 2100, with the best estimate being a 6% increase.

In most instances, hard erosion resistant shores will generally show no noticeable erosion in response to sea level rise, but the high-water line will still move landward according to the slope above the present high-water line. The Bruun rule can be applied to give a first estimate of possible erosion of 'soft' sandy beaches. In a case study, the combined effects were predicted of potential shoreline erosion with Bruun's rule, and higher wave runup from SLR with a 1-in-20 year sea storm on the Table Bay coast.

Some alternative methods to Bruun's rule to quantify cross-shore effects of SLR on sandy shores are discussed, but none were found to be suitable for general practical application in South Africa. In case of "mixed sandy/rocky" shorelines, a first estimate of the *maximum possible* potential recession due to SLR can still be made by means of the Bruun rule in many instances. A less conservative result may be derived by appropriately reducing the depth of closure when applying the Bruun rule. Alternatively, the "slope transferal" method may be applied to give an indication of the minimum recession expected due SLR effects. The most appropriate result for certain "mixed sandy/rocky" shorelines, would then be between this minimum value and the maximum value according to the Bruun rule applied with full closure depth.

Sandy shores located landward of extensive surf zone reefs are special situations, which should be thoroughly studied by determining the expected future wave conditions at the shoreline through wave modelling, or by considering depth limited wave conditions for the projected future seawater levels. Methods such as those described in Section 6.2 can then be employed to assess the potential erosion resulting from the SLR and amplified wave conditions.

Vegetation / wind-blown sand buffer areas

It has been determined that along the South African coast, the minimum practical width that is necessary to effectively trap wind-blown sediment ranges from about 10 m to 40 m, depending on circumstances. The low end of this range (10 m to 20 m) is applicable to beaches found along KZN, while the top end of the rage (30 m to 40 m) is applicable to areas such as the Cape.

Steep sandy slopes, coastal bluffs and cliffs

In assessing dune stability for dunes of relatively low height (<10 m from base to crest), it is usually sufficient to simply consider the angle of repose as the *maximum* potential stable slope angle. However, a *conservative* acceptable slope angle may be as mild as of 1:6 under stable conditions and even as gentle 1:10 under unstable conditions. For dunes of more than 10 m in height it is advisable to involve geotechnical engineers and to conduct a slip circle analyses, especially if the slope is steeper than 1:6. Alternatively, a conservative setback from the dune crest of at least two to three times the dune height is required.

The general *minimum additional* erosion setback provision for hard erosion resistant “stable” shorelines in South Africa is here recommended as follows: Cliffs - 15 m landward from the edge of the cliff; High rocky shores - 6 m landward from the top (crest) of steep slopes; and Low rocky shores - 30 m landward from the natural coastal vegetation line. (In extreme situations this should be checked against coastal flooding simulations to ensure adequate setback provision has been made.)

Soft rocks and erosive cliffs may be subject to significant coastal erosion within typical setback line planning horizons. The best practical means of assessing erosion setback requirements for such shorelines, is to analyse historic recession of these shorelines by means of, for example, aerial photography spanning at least 30 to 50 years. For soft rocks and erosive cliffs or bluffs of more than 10 m in height, it is advisable to involve geotechnical engineers and to conduct a slip circle analyses, especially if the slope is steeper than 1:6.

Estuaries and river mouths

A practical and robust means of assessing mouth dynamics and possible channel meandering, is to analyse historic shoreline changes in the vicinity of river mouths by means of, for example, aerial photography spanning at least 50 years. Erosion setback requirements in such locations are therefore strongly informed by historic data and information, and need to be conservative, as the maximum extent of progressive changes or long-term shoreline evolution is difficult to foresee or predict.

In South Africa three basic options are currently employed for delineating “setback lines” or “buffer areas” along estuarine reaches inland of the mouth. The assessment and determination of “setback lines” or “buffer areas” along estuarine reaches inland of the mouth, is a whole study area in itself, and is not focused on in this thesis. Depending on the detailed level of quantification required in comprehensive studies of individual estuaries, the expertise of geophysical estuarine specialists or engineers specializing in river/estuarine hydraulics/morphology should be employed to provide the necessary detailed assessment and robust “setback lines/buffer areas” recommendations. The best guidance should soon be forthcoming from the report “Managing South Africa’s estuaries for change, Version 1”, which is presently being drafted under the auspices of Department of Environmental Affairs (Van Niekerk *et al*, 2014).

<u>Ecological and social components of coastal setback lines</u>

Coastal development setback lines (or coastal “management” lines) also have to consider other important aspects, namely ecological and social components. <u>Ecological components</u> of coastal setback lines refer to ecologic, biodiversity, and environmental conservation aspects, and related ICM requirements. Due to reductions and losses of ecologically important zones in coastal areas, increased setback provisions are required to compensate or mitigate these impacts. This entails determining environment buffers required inland from the coastal processes setback to maintain a functional coastal ecosystem under present and future conditions. Such buffer zone widths depend on factors such as the recession rate, the importance and functional area requirements of the ecological zones, and social aspects. Provisions for ecological components of coastal setback lines should be informed by an environmental assessment based on the SANBI biophysical sensitivity layers. Critical Biodiversity Areas can be identified from biodiversity maps, which include information on proposed management of the area. Such assessments should be conducted by (or in full consultation with) biodiversity specialists or ecologists specializing in the coastal domain. The goal is a win-win (or “no regrets”) approach which serves both man and nature in enhancing both geophysical coastal protection (through vegetated dunes, mangroves, corals, wetlands, etc.) and serving environmental needs, which ultimately enables sustainable coastal development and tourism and long-term social benefits.

<u>Social components</u> of coastal setback lines include consideration of and making setback provisions for: Legal and zoning aspects, Public access, Aesthetic features, Heritage, and Shade. Appropriate provincial authorities (and e.g. heritage bodies, etc.), can provide reports and maps indicating issues/aspects/features related to the above that are located within the coastal zone. These social components include some amorphous aspects and typically require much wider consultation and public participation to resolve the issues. Thorough assessment of available information (e.g. reports, maps, etc.) together with such wide consultation should in most instances suffice. Ideally, or in special cases a site inspection accompanied by a relevant specialist is recommended in order to practically determine the buffer area that may be required for such aspects.

Before a coastal development setback line can indeed be promulgated, all the geophysical and biophysical components, as well as the socio-economic aspects have to be considered holistically and combined. In practice the coastal development setback line will always be landward of the coastal flooding and erosion setback provisions to allow space for wind-blown sand effects and other buffer zone properties. Finally the coastal processes setback provisions are combined with the ecological, social and coastal zone management (CZM) aspects/principles to derive the final coastal development setback line. In specific instances it may be practical to incorporate some ecological, social or CZM aspects into the coastal processes setback line provisions, i.e. all of the components do not necessarily have to be accumulated, if some of the provisions are already sufficient to serve simultaneous needs.

Chapter 9: Overall procedure for determining coastal development setback lines

9.1 Introduction

This chapter concerns logically combining all the setback line methodologies developed and aspects covered in the previous chapters to provide a complete procedure for determining setback lines. Thus, the basic components of coastal development setback lines are catalogued and a compilation of the steps required to practically determine coastal development setback lines is provided. Based on those developed in the previous chapters, recommended procedures and methods for conducting/completing each of the steps required to determine coastal development setback lines are then summarized. The procedures and methods are aimed as far as possible at meeting the ideals/norms for "good and proper" and appropriate setback line determination in South Africa.

The purpose of a coastal development setback line, in terms of the South African ICM Act (2008), should be kept in mind, which is to:

- Protect private and public coastal property, including natural environment;
- Demarcate safe areas, enable definition of areas at risk of being eroded or impacted by coastal processes, and enable the identification of infrastructure vulnerable to effects of SLR and inundation due to wave runup;
- Achieve conservation and sustainable development; and to
- Achieve other ICM considerations, e.g. bio-diversity, coastal conservation, etc.

An excellent example is shown in Figure 9.1 of an appropriate coastal development setback landward of a well-maintained natural foredune functioning as an effective buffer dune system.

Figure 9.1: Natures Valley, an excellent example of an appropriate development setback landward of a well-maintained natural foredune functioning as an effective buffer dune system *(Photo courtesy of DEA).*

9.2 Ideals for determination of coastal setback lines in South Africa

Some "hi-level standards" or "ideals" for determination of coastal setback lines in South Africa have been identified by the author as summarized below:

- Ideally a holistic, integrated and consistent (or "uniform") *approach* is required over all the provinces;
- Applied *methods* need to be comprehensive, robust and appropriate for setback lines in the South African context;
- The ("standard") methodology should be practical and implementable (on regional /national scale), thus affordable and efficient;
- Data inputs and techniques or procedures must be standardized as far as possible.

This clearly points out the need for guidelines (regarding methods), and even (as far as possible) some "norms and standards" for setback line determinations in South Africa.

Some of these are similar to the requirements listed by DEADP (2010), for "ideal" setback line methodologies:

- "The methodology should be applicable in all 4 coastal provinces. Therefore the methodology should consider conditions prevalent in all provinces;
- The methodology should be generally conservative in considering the accuracy of data, methods and climate change;
- The methodology should not rely on excessively expensive and time-consuming data collection and should minimise costly specialist expertise, over and above the essential coastal processes expertise required;
- The methodology must represent international best practice;
- The methodology must be legally defendable and must withstand legal scrutiny;
- The methodology must ideally be reproducible, i.e. if conducted by another professional a similar result should be obtained."

The overall procedure and methods recommended in the following sections of this chapter attempt as far as possible to meet the "norms" listed in the foregoing section.

9.3 Necessary components of coastal development setback lines

9.3.1 Catalogue of basic components required for setback lines

Basic components of "coastal processes" setback lines

Chapter 5 (coastal flooding levels), Chapter 6 (shoreline changes and coastal erosion), Chapter 7 (additional protection provided by dunes), and parts of Chapter 8 (other components and aspects of setback lines – Sections 8.1 to 8.5) have all dealt with the abiotic coastal processes (which is the focus of this thesis). "Coastal processes" setback lines, which mainly address safety and protection of property from these abiotic physical coastal/marine processes/"impacts" should therefore include the following basic components:

A. *Setback provision for coastal flooding, inundation, direct wave impacts, based on extreme water levels and wave runup determined according to the findings of Chapter 5;*

B. *Setback provisions for shoreline changes and coastal erosion determined according to the findings of Chapter 6. This comprises of a setback provision for short-term shoreline variation, e.g. erosion due to storm waves, which includes accounting for dunes where applicable (as per Chapter 7), as well as a setback provision for long-term coastline changes (shoreline location trends);*

C. *Setback provisions for additional physical coastal/marine processes/"impacts" aspects determined according to the findings of Chapter 8, which comprises of provisions for:*

    i. potential climate change effects, primarily sea level rise and possibly wave height increase (as per Sections 8.2 and 8.7);
    ii. wind-blown sand (vegetation buffers) (as per Sections 8.3 and 8.7);
    iii. bluff/dune/cliff instability (as per Sections 8.4 and 8.7);
    iv. estuary or river mouth dynamics (as per Sections 8.5.1 and 8.7);

D. *"Setback lines/buffer areas" along estuarine reaches inland of the mouth.* This aspect is listed separately because it in fact entails a whole separate procedure and methodologies (as per Sections 8.5.2 and 8.7), which is not focussed on in this thesis. *The coastal development setback line (along the seashore) must finally be merged with the "setback lines/buffer areas" along estuarine reaches inland of the mouth (along the river course).* There should be no discontinuity or abrupt offsets between the coastal and estuarine setback lines, so the lines have to be merged smoothly whilst maintaining the integrity within each domain. This is, therefore, only an issue in the vicinity of the mouth of the estuary, where the coastal setback lines (along the seashore on both sides of the mouth) meet with the estuarine "setback lines" (along both banks of the river course).

Basic components of ecological and social components of coastal setback lines

The above components of setback lines mainly deal with the geophysical coastal-marine processes, dynamics and components of coastal setback lines (which, as mentioned, are indeed the focus of this thesis). As discussed in Chapter 8, coastal development setback lines (or coastal "management" lines) also have to consider other important aspects, which have been grouped under the main headings of ecological and social components of coastal setback lines (Section 8.6). These aspects therefore require setback provisions determined according to the findings of Chapter 8, which comprises of:

E. *Ecological components*

Ecological components of coastal setback lines refer to ecologic, biodiversity, and environmental conservation aspects, and related ICM requirements, and entails determining respective environment buffers (Section 8.6.1).

F. Social components of coastal setback lines

These include consideration of and making setback provisions for: legal and zoning aspects, public access, aesthetic features, heritage preservation, and shade issues (Section 8.6.2).

Combining coastal processes setback lines, coastal zone management issues and estuarine setback lines

G. Combining geophysical and biophysical components with socio-economic aspects

Before a coastal development setback line can indeed be promulgated, all the geophysical and biophysical components, as well as the socio-economic aspects have to be considered holistically and combined (as discussed in Section 8.6.3). This component usually requires wider consultation (including public participation) and is beyond the focus of this thesis.

9.3.2 Procedure for determining coastal development setback lines

Based on the setback line methodologies developed and aspects covered in the previous chapters, and the catalogue of required basic components of setback lines as provided in Section 9.3.1, an overall procedure for determining coastal development setback lines is recommended. A logical compilation of the steps required to determine the coastal development setback line is provided in Figure 9.2, also considering that some steps provide the required input for certain other steps (e.g. the wave climate is required for both erosion and coastal flooding determination). Note, that the full (formal) process of determining and promulgating coastal setback lines includes additional steps/components such as stakeholder engagement, public participation, dissemination, etc. which are not included in the compilation depicted in Figure 9.2.

1. **Understand the study area** (Chapters 2, 4)
   - Obtain an understanding of the coastal processes and biophysical aspects of the study area as well as in the larger "regional" context
2. **Determine long-term shoreline location trends** (Chapter 6)
   - Assess historic shoreline variations and trends
   - Determine setback provision for long-term shoreline location trends
3. **Determine the wave climate** (Chapters 2, 4)
   - Determine offshore wave climate - present and future
   - (Determine inshore wave climate - present and future)
4. **Determine the erosion setback** (Chapter 6)
   - Determine setback provision for short-term erosion, account for dunes
   - Determine setback provision for sea level rise (SLR)
5. **Determine the coastal flooding elevations** (Chapter 5)
   - Determine extreme seawater levels, include SLR scenarios
   - Model wave runup levels
   - Determine coastal flooding elevations
6. **Determine the setback provisions for additional aspects** (Chapter 8)
   - Determine vegetation buffers for wind-blown sand
   - Determine setback provisions for bluff/dune/cliff instability
   - Determine setback provisions for estuary/river mouth dynamics
7. **Determine setback lines/buffer areas along estuaries** (Chapter 8)
   - Determine setback lines/buffer areas along estuarine reaches inland of the mouth.
8. **Determine buffers / setback provisions for ecological components** (Chapter 8)
   - Determine environment buffer areas for ecology, biodiversity, environmental conservation aspects, and related ICM requirements
9. **Determine buffers / setback provisions for social components** (Chapter 8)
   - Determine provisions for: legal and zoning aspects, public access, aesthetic features, heritage, and shade
10. **Combine coastal processes setback lines & coastal zone management issues**
    - Combine geophysical & biophysical components with socio-economic aspects
    - Merge setback lines/buffer areas along estuaries with coastal setback lines

Figure 9.2: Compilation of the steps required to determine the coastal development setback line.

9.3.3 Recommended methods for completing the steps required to determine setback lines

Recommended procedures and methods for conducting/completing each of the steps required to determine coastal development setback lines are summarized in the following sections. This basically follows the 10 main steps as compiled in Figure 9.2 and provides the details of the sub-steps needed to complete each main step. These recommended procedures and methods are based on the methods developed in, and findings from Chapters 5 to 8, as follows: the two most key aspects are coastal flooding (Chapter 5) and erosion (Chapter 6) as addressed in Steps 2 to 5 of the 10-step procedure outlined in Figure 9.2, while the "dune" method (Chapter 7) is applicable in some instances (part of Step 4). The additional "coastal processes" aspects are addressed in Step 6, based on the discussions and findings in Sections 8.1 to 8.5 (informed by Chapters 2 and 4). The remaining components of determining the complete coastal development setback line, entail the estuarine, ecological and social components, thus Steps 7, 8 and 9 respectively, while the final step (Step 10) entails combining all the different components. These latter components (Steps 7 to 10) are based on Sections 8.5 and 8.6 (informed by Chapters 2 and 4), but are not the focus of this thesis and therefore not addressed in detail. To cater for a nation-wide approach, some aspects of the guidelines, mainly the additional "coastal processes" aspects in Step 6, are necessarily somewhat general (and standard), as they are largely based on the most relevant findings from the literature (Chapter 2). More details on these aspects are however provided in Sections 8.1 to 8.5.

It should be noted that in specific circumstances, the recommended procedures and methods provided below do not preclude the application of alternative conventional methods in finer scale detailed investigations of small study areas, e.g. for large existing or planned infrastructure developments.

Step 1: Understand the study area

- Obtain an understanding of the coastal processes and biophysical aspects of the study area as well as in the larger "regional" context.

The first and very important step/requirement of determining coastal setback lines is to obtain a good understanding of the coastal processes and dynamics at the study area as well as in the larger "regional" context. Coastal development setback lines should be determined by studying all available information regarding the geography, coastal dynamics, beach characteristics, wave regime, long-term shoreline evolution, eustatic rise in sea level, aeolian sediment dynamics and the characteristics of the foredune (should such a dune exist). Essential information for determining a setback line includes good topographical data of the shoreline and backshore areas, such as can be provided by means of, for example, a conventional topographical or LiDAR survey. Investigation of all these aspects aid the long-term protection (or sustainability) of developments and help to keep the littoral active zone free from impacts due to unwisely sited infrastructure or developments.

Step 2: Determine long-term shoreline location trends

- Assess historic shoreline variations and trends.

Long-term shoreline changes can be quantified by considering the variation in shoreline location over an extended period. Analysis of vertical aerial photography is the usual means of doing this and such imagery (including for example suitable satellite images) is available for virtually the entire South African coast. (Analyses of beach topographical surveys should be included if such data is available.) The identification of pertinent features, processes and coastline changes (shoreline, vegetation line, etc.) from aerial photography also assists in understanding the study area (Step 1).

- Determine setback provision for long-term shoreline location trends.

If a significant eroding trend is apparent in the shoreline location, a conservative estimate of the erosion rate is extrapolated for the stipulated/chosen setback “design lifetime”/planning period, usually 50 or 100 years. This constitutes the setback provision for long-term shoreline location trends, which is later added to the erosion setback provision. More details are provided in Section 6.2.

Step 3: Determine the wave climate

- Determine offshore wave climate - present and future.

The present offshore wave climate at deep sea locations around the South African coast can be determined by using NCEP hind cast wave data (NCEP 2013), from the NOAA/NCEP WAVEWATCH III Global Model (Tolman *et al* 2002). (Alternatively, in some areas the wave climate may be derived from nearshore recordings off some of the major South African ports, potentially available from TNPA (Transnet National Ports Association of South Africa).) Currently an appropriate scenario for future wave climate off the South African coast is a 6% to 10% increase in wave height by 2100, with the best estimate being a 6% increase (as per Section 8.2.2).

- Determine inshore wave climate - present and future.

To determine the inshore wave climate along the study area, mathematical wave modelling needs to be conducted using a state-of-the-art refraction model. Hydrodynamic wave modelling (SWAN, Booij *et al*, 1999) is, for example, used to transform the offshore wave data to inshore conditions. Potential climate change effects can be included by simply assuming a wave height increase in accordance with the scenario given above. (A more correct procedure would be to account for both a rise in sea level and possibly offshore wave climate changes in modelling the future inshore wave conditions. This elaborate procedure is however only considered to be appropriate in some detailed investigations of small study areas. In general the relatively small additional accuracy in potential future inshore wave conditions is not considered to be significant keeping in mind for example the large uncertainties in future scenarios for offshore wave climate.)

Step 4: Determine the erosion setback

- Determine setback provision for short-term erosion, account for dunes.

Topographic beach survey data is analyzed to determine short-term variability (as per Section 6.2.2). This data can be used to predict the maximum short-term erosion (event) over a selected period (say 50 years), based on the Normal Model (Equation 6.1). Where no shoreline change data other than aerial photography is available, an initial estimate of short-term coastal erosion can be made by judiciously applying the Normal model. There should be virtually no area along the South African coast for which at least 10 different images are not available, which is recommended as the minimum number to provide some confidence in the analysis.

Alternatively, the formulation derived in Equation 6.6 for predicting cross-shore erosion distance due to storm waves, i.e. the Parametric shoreline erosion model can be used. The required input parameters are the offshore wave height and period (H_0 and T_p), the sediment grain size (D_{50}) and the bottom slope to 20 m depth (tan_{20}). Ideally, both methods can be used to compare and resolve or verify the results. More details are provided in Chapter 6. The potential climate change effects of a possible wave height increase on the short-term erosion can also be estimated by means of the Parametric model.

A third, more elaborate procedure may be considered for some detailed investigations of small study areas. The conventional/existing cross-shore sediment transport or morphology models used for such purposes quantify the local transport rates and time-dependent beach profiles, but are typically "data-hungry" and require significant calibration. Thus they are largely suited to such detailed studies of small areas and face substantial practical difficulties for application to large study areas.

To account for the additional protection provided by dunes in some instances, the Dune model can be applied (i.e. the generalized dune volume versus setback line distance relationship as determined in Equation 7.1). The "Dune" setback line methodology for large study areas is explained in detail in Section 7.5.1.

- Determine setback provision for sea-level rise (SLR).

The Bruun rule can be applied to give a first estimate of possible erosion of 'soft' sandy beaches due to SLR. The recommended scenarios of SLR to apply for 2100 are: 0.5 m (low), 1 m (medium), and worst-case of 2 m (high). The best estimate ("medium") scenarios for 2030 and 2050 are 0.15 m and 0.35 m, respectively (as per Section 8.2.1).

Hard erosion resistant shores will generally show no noticeable erosion in response to sea level rise, but the high-water line will still move landward according to the slope above the present high-water line. In case of "mixed sandy/rocky" shorelines, a first estimate of the maximum possible potential recession due to SLR can still be made by means of the Bruun rule in many instances. A less conservative result may be derived by appropriately reducing the depth of closure when applying the Bruun rule. Alternatively, the "slope transferal" method may be applied to give an indication of the minimum recession expected due to SLR effects. The most appropriate result for certain "mixed sandy/rocky" shorelines, would then be between this minimum value and the maximum value according to the Bruun rule applied with full closure depth. More details are provided in Section 8.2.

Step 5: Determine the coastal flooding elevations

- Determine extreme seawater levels, include SLR scenarios.

The tidal ranges for the South African coast are summarized in Table 5.1. Extreme South African seawater levels excluding tides (thus mainly due to wind and hydrostatic setup) have been analysed for all of the South African tidal stations, and the results (i.e. residuals for various return periods) are summarized in Table 5.2. The scenarios for SLR to be included in the seawater levels, are as recommended in the previous Step 4. Estimates are provided in Section 5.2.5 of extreme values for realistic combinations of all the inshore seawater level components, as applicable to each South African coastal region. A first-order coarse storm surge level assessment for the South African coastal regions indicating the relative coastal flooding levels of the different South African coastal regions is provided in Section 5.2.5.

- Model wave runup levels.

The wave runup models of Nielsen and Hanslow (1991) and Mather *et al* (2011) are the best of the available models and are adequate for application in South Africa, but should be used with certain adaptations as recommended here. Overall, the Nielsen and Hanslow model is the most suitable; the best results will be obtained with significant wave heights determined at about 20 m depth or less and then “reverse shoaled” to give the equivalent deep-water wave heights as input. The other inputs required are the wave period, the beach slope, and the still water level.

Where only deep-water heights are known, or where data is lacking on the beach slope, the Mather *et al* model can be applied. The inputs required are the deep-water wave height, the distance to the 15 m contour and the still water level. The value of coefficient C should be set at 7.5 in open coast locations and even in semi-exposed locations. In well sheltered locations, the value of coefficient C should provisionally be set at 5. Ideally, both methods can be used to compare and resolve or verify the results.

Potential climate change effects, primarily sea level rise, would already be included in the extreme seawater levels as discussed above. The potential climate change effects of a possible wave height increase on the runup levels can also be included directly in both models.

Wave runup heights on rocky shorelines can be determined by means of the method in the Eurotop manual (Pullen *et al*, 2008).

- Determine coastal flooding elevations.

The results from the wave runup models already include all the necessary sea-level and runup components, thus directly yield the coastal flooding elevations.

If coastal flooding elevations are found to exceed the dune elevation at some point, then overtopping of such locations and potential flooding of low-lying backshore areas should be considered (DEADP, 2010). The overtopping can be assessed by means of the EurOtop method (Pullen *et al*, 2008; an interactive tool is also available at www.overtopping-manual.com).

Step 6: Determine the setback provisions for additional aspects

- Determine vegetation buffers for wind-blown sand.

It has been determined that along the South African coast, the minimum practical width that is necessary to effectively trap wind-blown sediment ranges from about 10 m to 40 m, depending on circumstances. The low end of this range (10 m to 20 m) is applicable to beaches found along KZN, while the top end of the rage (30 m to 40 m) is applicable to areas such as the Cape.

- Determine setback provisions for bluff/dune/cliff instability.

In assessing dune stability for dunes of relatively low height (<10 m from base to crest), it is usually sufficient to simply consider the angle of repose as the maximum potential stable slope angle. However, a conservative acceptable slope angle may be as mild as 1:6 under stable conditions and even as gentle 1:10 under unstable conditions. For dunes of more than 10 m in height it is advisable to involve geotechnical engineers and to conduct a slip circle analyses, especially if the slope is steeper than 1:6. Alternatively, a conservative setback from the dune crest of at least two, but preferably three times the dune height is required.

The general minimum additional erosion setback provision for hard erosion resistant “stable” shorelines in South Africa is here recommended as follows: cliffs - 15 m landward from the edge of the cliff; high rocky shores - 6 m landward from the top (crest) of steep slopes; and low rocky shores - 30 m landward from the natural coastal vegetation line. (In extreme situations this should be checked against coastal flooding simulations to ensure adequate setback provision has been made.)

The best practical means of assessing erosion setback requirements for soft rock and erosive cliff shorelines, is to analyse historic recession of these shorelines by means of, for example, aerial photography spanning at least 30 to 50 years. For soft rocks and erosive cliffs or bluffs of more than 10 m in height, it is advisable to involve geotechnical engineers and to conduct a slip circle analyses, especially if the slope is steeper than 1:6.

- Determine setback provisions for estuary/river mouth dynamics.

A practical and robust means of assessing mouth dynamics and possible channel meandering, is to analyze historic shoreline changes in the vicinity of river mouths by means of, for example, aerial photography spanning at least 50 years. Erosion setback requirements in such locations are therefore strongly informed by historic data and information, and need to be conservative, as the maximum extent of progressive changes or long-term shoreline evolution is difficult to foresee or predict.

Step 7: Determine setback lines/buffer areas along estuaries

- Determine setback lines/buffer areas along estuarine reaches inland of the mouth.

In South Africa three basic options are currently employed for delineating “setback lines” or “buffer areas” along estuarine reaches inland of the mouth. *The assessment and determination of “setback lines” or “buffer areas” along estuarine reaches inland of the mouth, is a whole study area in itself, and is not focused on in this thesis.* Depending on the detailed level of quantification required in comprehensive studies of individual estuaries, the expertise of geophysical estuarine specialists or engineers specializing in river/estuarine hydraulics/morphology should be employed to provide the necessary detailed assessment and robust “setback lines/buffer areas” recommendations. *The best current guidance should soon be forthcoming from the report “Managing South Africa’s estuaries for change, Version 1”, which is presently being drafted under the auspices of Department of Environmental Affairs (Van Niekerk et al, 2014).*

Step 8: Determine buffers / setback provisions for ecological components of coastal setback lines

- Determine environment buffer areas for ecology, biodiversity, environmental conservation aspects, and related ICM requirements.

The foregoing Steps 2 to 7 of setback lines mainly deal with the geophysical coastal-marine processes, dynamics and components of coastal setback lines (which are indeed the focus of this thesis). Coastal development setback lines (or coastal “management” lines) also have to consider other important aspects, which have been grouped under the main headings of ecological and social components of coastal setback lines (Section 8.6).

Ecological components of coastal setback lines refer to ecologic, biodiversity, and environmental conservation aspects, and related ICM requirements. Due to reductions and losses of ecologically important zones in coastal areas, increased setback provisions are required to compensate or mitigate these impacts. This entails determining environment buffers required inland from the coastal processes setback to maintain a functional coastal ecosystem under present and future conditions. Such buffer zone widths depend on factors such as the coastal recession rate, the importance and functional area requirements of the ecological zones, and social aspects.

Provisions for ecological components of coastal setback lines should be informed by an environmental assessment based on the SANBI biophysical sensitivities layers. Critical Biodiversity Areas can be identified from biodiversity maps, which include information on proposed management of the area. Such assessments should be conducted by (or in full consultation with) biodiversity specialists or ecologists specializing in the coastal domain. The goal is a win-win (or "no regrets") approach which serves both man and nature in enhancing both geophysical coastal protection (through vegetated dunes, mangroves, corals, wetlands, etc.) and serving environmental needs, which ultimately enables sustainable coastal development and tourism, and long-term social benefits.

Step 9: Determine buffers / setback provisions for social components of coastal setback lines

- Determine provisions for: legal and zoning aspects, public access, aesthetic features, heritage, and shade.

Social components of coastal setback lines include consideration of and making setback provisions for:

i. Legal and zoning aspects;
ii. Public access;
iii. Aesthetic features;
iv. Heritage;
v. Shade.

Appropriate provincial authorities (and e.g. heritage bodies, etc.), can provide reports and maps indicating issues/aspects/features related to the above that are located within the coastal zone. These social components include some amorphous aspects and typically require much wider consultation and public participation to resolve the issues. Thorough assessment of available information (e.g. reports, maps, etc.) together with such wide consultation should in most instances suffice. Ideally, or in special cases a site inspection accompanied by a relevant specialist is recommended in order to practically determine the buffer area that may be required for such aspects.

Step 10: Combine coastal processes setback lines & coastal zone management issues

- Combine geophysical and biophysical components with socio-economic aspects.

Before a coastal development setback line can indeed be promulgated, all the geophysical and biophysical components, as well as the socio-economic aspects have to be considered holistically and combined. In practice the coastal development setback line will always be landward of the coastal flooding and erosion setback provisions to allow space for wind-blown sand effects and other buffer zone properties. Finally the coastal processes setback provisions are combined with the ecological, social and coastal zone management (CZM) aspects/principles to derive the final coastal development setback line. In specific instances it may be practical to incorporate some ecological, social or CZM aspects into the coastal processes setback line provisions, i.e. all of the components do not necessarily have to be accumulated, if some of the provisions are already sufficiently severe to accommodate simultaneous needs.

(DEA&DP (2010) state that after a site visit with a biodiversity specialist to ensure that the status of the relevant biodiversity map is up to date, a limited development area can be assigned. This may be acceptable in some instances, but may certainly also be problematic if no strong argument/reason is provided for such "concessions" to development to be made (i.e. allowing the development to be placed within the environmental zone/buffer). The environmental issues/objectives are not necessarily of lesser importance than physical damage to structures for which case such "concessions" are usually not made (i.e. the development is usually not allowed to be placed within the coastal processes setback zone). The same argument could be made regarding other issues such as aesthetics and heritage provisions possibly allowing for limited development in the same area as these assets are located, which should ideally not be the norm.)

The sketch provided in Figure 9.3 (from Barwell, 2011), provides a graphical illustration of how all the different components are combined in setting the coastal setback line.

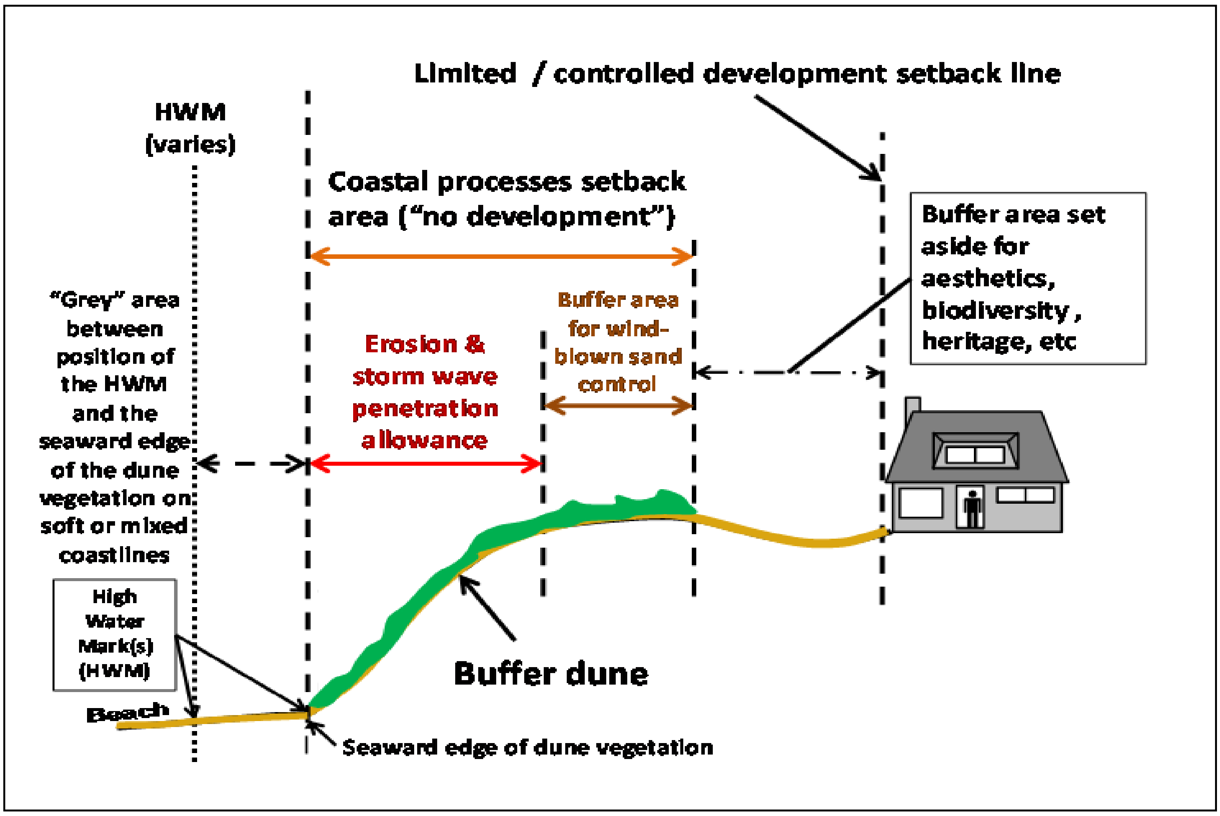


Figure 9.3: Illustration of how all the different components are combined in setting the coastal setback line. *(from Barwell, 2011)*

- Merge setback lines/buffer areas along estuaries with coastal setback lines.

As mentioned, the coastal development setback line (along the seashore) must finally be merged with the "setback lines/buffer areas" along estuarine reaches inland of the mouth (along the river course). There should be no discontinuity or abrupt offsets between the coastal and estuarine setback lines, so the lines have to be merged smoothly whilst maintaining the integrity within each domain. This is therefore only an issue in the vicinity of the mouth of the estuary, where the coastal setback lines (along the seashore on both sides of the mouth) meet with the estuarine "setback lines" (along both banks of the river course). The procedure is relatively straightforward once the coastal and estuarine "setback lines" have been mapped. Where the coastal lines along the seashore on both sides of the mouth intersect with the estuarine lines along both banks of the river course, they are joined (whilst ensuring that the merged line follows the position most distant/inland from both the coastal and estuarine waterlines). Use of a GIS system (/software) facilitates this mapping and merging of the lines, which also enables easy graphical display of the lines (and dissemination of the results).

9.4 Primary data requirements and sources

The lists compiled here are not intended to be exhaustive or fully inclusive, but indicate most of the major data requirements for determination of setback lines and some typical sources of such data.

Primary data/information required for determination of setback lines include:

- Coastal topography (including dune and beach profile data);
- Bathymetry (including nearshore and ideally inshore, although inshore bathymetry is usually only available for prior large projects);
- Sediment characteristics (especially grain size data and typology);
- Present and future metocean climate (wave and wind regime, tides, currents, atmospheric pressures, sea level, and future metocean scenarios);
- Historic shoreline changes (including high-water line, vegetation line, dune/vegetation areas and aeolian sediment pathways);
- Estuarine mouth dynamics and historic channel migration configurations;
- Spatial Development Frameworks (from Municipalities);
- Coastal geography, geologic/geomorphology information;
- Biodiversity maps provided by nature conservation bodies (e.g. Cape Nature) and SANBI;
- Cadastral boundaries, coastal protection zones, Municipal/Town planning zones, military/other special use areas, special management areas, etc.; and
- Reports and maps indicating heritage sites.

In terms of the data/inputs required for the coastal processes setback, some of the most onerous requirements are: coastal topography, inshore wave conditions, historic shoreline changes, and potentially inshore bathymetry, although the coarser SAN bathymetry data is mostly sufficient.

Sources of such information/data include:

- Offshore wave climate: NCEP hind cast wave data (NCEP 2013, from NOAA/NCEP WAVEWATCH III Model);

- Nearshore wave data off South African ports: TNPA (Transnet National Ports Association of South Africa);
- Aerial photographs, ortho-photographs: Surveyor General, local and provincial authorities;
- Topographic surveys, aerial photogrammetry, LiDAR: local and provincial authorities, surveying companies (land or air);
- Remote sensing, satellites: Google Earth, RS companies;
- Geophysical GIS data: online DWAF GIS data/layers, Agricultural Research Council, etc;
- Bathymetric data and charts: South African Navy Hydrographic office, TNPA, potentially prior large projects;
- Tides, seawater levels: South African Navy Hydrographic office;
- Wind data: South African Weather Bureau, TNPA at the ports, nearby major airports;
- Barometric data: South African Weather Bureau;
- Estuarine mouth dynamics and historic channel migration configurations: CSIR Estuarine Green Reports Series" (for example Duvenhage and Morant 1984, CSIR 1987b, etc.);
- Nearshore wind (and current) data off South African ports: TNPA (Transnet National Ports Association of South Africa).

The use and availability of LiDAR data providing detailed coastal topography of large areas, and to assess shoreline position and change (e.g. Stockdon *et al* 2002) was a "privilege" usually not available in South Africa, but is now becoming much more prevalent locally. Minimum data standards and specifications for coastal LiDAR data, suitable for setback lines are provided in Lück-Vogel *et al*, 2014.

9.5 Application and demonstration of the proposed procedure and methods to determine setback lines

Case studies were conducted to demonstrate the systematic set-by-step application of the complete procedure and methods to determine setback lines, in accordance with the recommendations as provided in Section 9.3 (following the 10 main steps as compiled in Figure 9.2). Two study areas were selected, namely one along the open coast at Richards Bay in northern Kwazulu-Natal on the Indian Ocean seaboard, and the other along the semi-sheltered Table Bay coast on the Atlantic Ocean seaboard along the southwestern African coast (Figure 6.3). By selecting these two diverse study areas, the application of the procedures to a variety of environments that are typical of the South African coast are demonstrated. More specifically, the setback lines were determined along the southern portion of the Table Bay coast from Salt River Mouth to the Milnerton area (Figure 9.4), and the Richards Bay northern beaches area up to a distance of 2.5 km north of the northern breakwater (Figure 6.3b).

Figure 9.4: The southern Table Bay setback line study area *(Google Earth image)*

Step 1: Understand the study area

A good understanding of the coastal processes and dynamics at the two study areas has been obtained, as is evidenced by several previous references made to these two areas in Chapters 2, 5, 6 and 7. This includes the larger "regional" context, while a brief description is also provided in Section 2.2 of the morphological characteristics of the two coastal regions, namely the South West Coast and the SA East Coast of South Africa, within which the study areas lie.

Step 2: Determine long-term shoreline location trends

- Assess historic shoreline variations and trends, and determine setback provision for long-term shoreline location trends.

Based on shoreline change data contained in DEAD&P (2010) from analyses of 9 sets of aerial photographs spanning the period from 1945 to 2008, the average recession rate in southern Table Bay (from Salt River Mouth to the Milnerton area, Figure 9.4) is calculated to be 0.49 m/a. Smith *et al* (2000) found an average recession rate of about 0.4 m/a for this area based on topographic surveys spanning the period 1967 to 1999. The topographic survey data is considered to be more accurate than the aerial photography data. Northern Table Bay from Table View/Rietvlei northwards to Blaauwberg Rocks did not exhibit a long-term shoreline erosion trend. Thus, the setback provision for long-term shoreline location trends in Southern Table Bay over a planning period of 50 and 100 years is calculated to be 20 m and 40 m respectively, while no such provision is needed for Northern Table Bay. As mentioned, setback lines can be determined for a range of planning horizons, but in accordance with Section 8.1 the most appropriate planning horizon in this case was considered to be 100 years.

Previous analyses of about 30 years of survey data of the northern beaches of Richards Bay have indicated an average long-term erosional trend of approximately 1 m per year (CSIR, 2005). In this area the setback provision for long-term shoreline location trends over a planning period of 50 and 100 years is thus calculated to be 50 m and 100 m respectively.

Step 3: Determine the wave climate

- Determine offshore wave climate - present and future.

As mentioned in Section 5.3, an analysis was previously undertaken by Rossouw (Rossouw and Theron, 2012) of the offshore wave climate at deep sea offshore locations around the South African coast (using NCEP hind cast wave data from the NOAA/NCEP WAVEWATCH III Global Model). Currently an appropriate scenario for future wave climate off the South African coast is a 10% increase in wave height by 2100 (as per Section 8.2.2). A summary of the pertinent extreme wave conditions off Richards Bay and off Table Bay is provided in Table 9.1.

Table 9.1: Summary of the extreme offshore wave conditions off Table Bay and Richards Bay.

Location	Return period (year)	Significant wave height (m)	Climate Change: H_0 + 10% (m)
Table Bay	50	12.6	
	100	13.2	
	200	13.8	
Richards Bay	50	9.3	10.3
	100	10.0	11.0

- Determine inshore wave climate - present and future.

The determination of the inshore wave climate along much of the South African coast by Rossouw *et al* (2014), who conducted hydrodynamic wave modelling to transform the offshore wave data to inshore conditions, has been discussed in Section 2.2.3. The numerical modelling enabled the derivation of the nearshore wave climates for locations at about 500 m intervals approximately along the 15 m isobath in each modelled area, which included both Table Bay and Richards Bay areas. Based on the modelled nearshore data, extreme wave heights were also determined (Rossouw *et al*, 2015). By means of this procedure the significant wave heights were determined for various return periods ranging from 1-in-1 year to 1-in-100 years. As an example, the 1-in-1 year and 1-in-30 year wave heights along the 15 m isobath in the Richards Bay coastal area are depicted in Figure 9.5 (Rossouw *et al*, 2014). Potential climate change effects can be included by simply assuming a wave height increase in accordance with the scenario given in Table 9.1, as is demonstrated in the steps to follow.

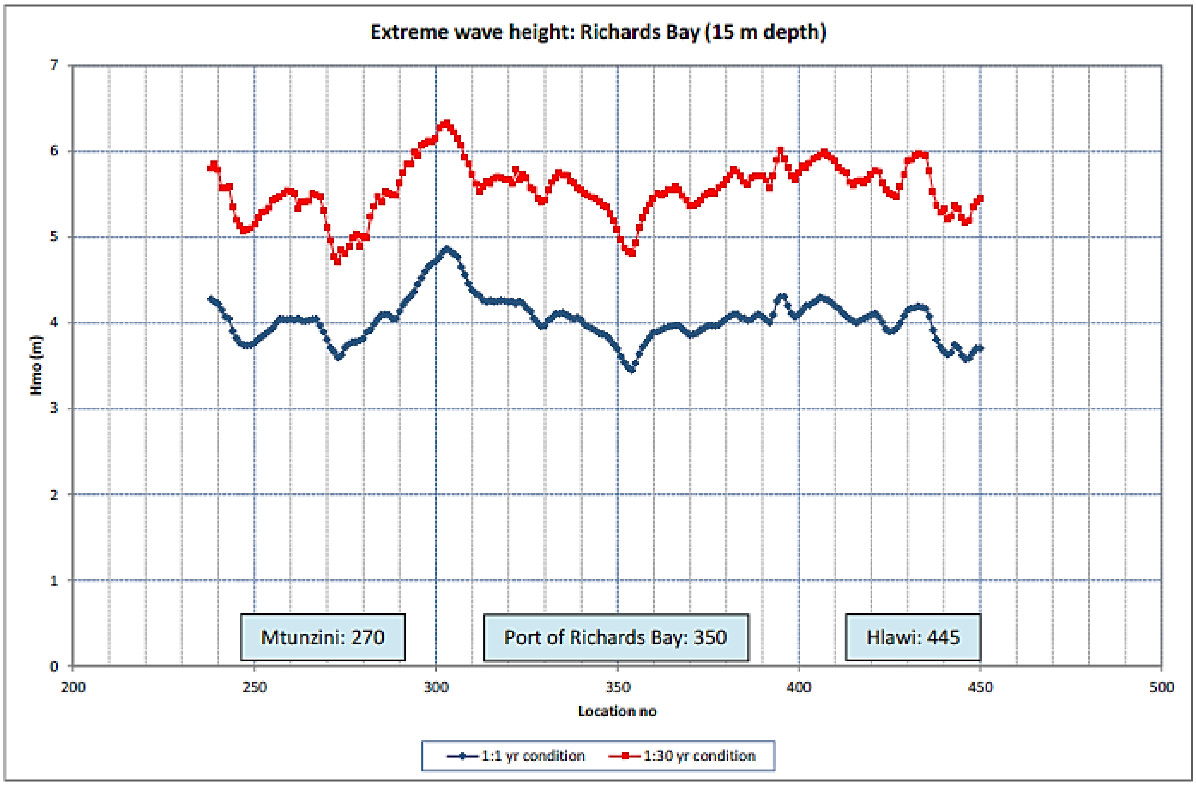


Figure 9.5: Wave heights along the 15 m isobath at Richards Bay for example return periods. *(Rossouw et al, 2014)*

Step 4: Determine the erosion setback

- Determine setback provision for short-term erosion.

<u>Table Bay</u>

Both the Normal (Equation 6.1) and Parametric (Equation 6.6) erosion prediction methods were applied to the area covered by the Table Bay wave runup studies as discussed in Section 5.3, specifically the southern portion of Table Bay (Figure 9.4; from Salt River Mouth to the Milnerton area). The slope, grain size, water levels and wave data were determined for this area as described before in Section 5.3. The predicted erosion results for the two models are indicated in Table 9.2. The results from the two models compare very well, with a difference of only 3.3 m in predicted shoreline erosion for the 1-in-100 year return period. Although both models have produced very similar and realistic results, the more conservative value will be applied in determining the setback line (following the precautionary principle according to the objectives of the South African ICM Act).

Table 9.2: Determination of setback provisions for short-term shoreline erosion.

Location	**Return period (year)**	**Predicted erosion Normal model (m)**	**Predicted erosion Parametric model (m)**	**Predicted erosion Parametric model & Climate Change (Hs + 10% (m)**
Table Bay	50	39	36	41
	100	**41**	**37**	**43**
	200		40	
Richards Bay	50	62	70	78
	100	**64**	**74**	**84**

To illustrate how the potential effects of climate change can be incorporated into the erosion predictions, a scenario of a 10% increase in extreme wave heights was simulated by means of the Parametric model. The results are also indicated in Table 9.2, for example, indicating a 6 m increase in predicted erosion for the 1-in-100 year return period (from 37 m to 43 m). To illustrate how the potential erosion predictions can be made for longer recurrence intervals, wave heights were determined for a 200 year return period, as provided in Table 9.1. Erosion predictions were then made for these 1-in-200 year wave heights by means of the Parametric model. These results are also included in Table 9.2 for the 200 year return period, indicating a relatively small increase of 3 m in predicted erosion (from 37 m to 40 m) for southern Table Bay (which is in accordance with the small wave height increase).

Richards Bay

Both of the erosion models were similarly applied to the Richards Bay study area (Figure 6.3b), namely the area up to a distance of 2.5 km north of the northern breakwater (covered by profiles 1 to 6 listed in Table 6.4). The slope, grain size and shoreline variation data was determined for this area as described before in Section 6.2.2. The predicted erosion results for the two models are also indicated in Table 9.2. The results of the two models compare relatively well, with a difference of 10 m in predicted erosion for the 1-in-100 year return period (i.e. 64 m and 74 m respectively). As before, the more conservative of the two results will be applied further in determining the setback line. Similar to the Table Bay study area, the potential effects of climate change were incorporated into the erosion predictions by means of the Parametric model, by simulating a scenario of a 10% increase in extreme wave heights. These results are also indicated in Table 9.2, indicating a 10 m increase in predicted erosion for the 1-in-100 year return period (from 74 m to 84 m).

To quantify the additional (potentially unaccounted for) protection provided by dunes, the Dune model (i.e. the generalized dune volume versus setback line distance relationship as determined in Equation 7.1) can be applied in some instances. There are basically three possible scenarios that may occur, which relate to the location of the dune on the cross-shore profile, as explained in detail in Section 7.5.1. According to the definition of these scenarios and the local situation at the Table Bay and Richards Bay study areas, Scenario A applies to both of the study sites, as the dunes are located within the envelope of recorded shoreline locations. Thus the effects of these dune are inherent in the recorded data which carries through into the results (from the Normal model), and explicit *additional* consideration of the effect of dunes is not accounted for in the determination of the erosion setback (as per Section 7.5.1). Thus, no further adjustment (reduction) of the erosion setback distance is made on account of the dunes found at the Table Bay and Richards Bay study areas.

- Determine setback provision for sea-level rise (SLR).

Based on "Bruun's erosion rule" and SANHO bathymetric charts of Table Bay, the potential shoreline erosion due to sea level rise was investigated, for the scenario of 1.0 m rise by 2100 (as per Section 5.2.4). The Bruun model predicts that the areas along the northern Table Bay coastline with a steeper nearshore slope will erode by about 187 m for this scenario, while the southern areas with relatively milder or flatter nearshore slopes are predicted to erode by about 41 m (Table 9.3). The average potential erosion for the 1 m sea level rise scenario is about 103 m. In the Table Bay situation, where hard structures (including revetments) are found in some shoreline locations, these may significantly reduce or virtually halt such horizontal shoreline erosion (assuming that they remain intact).

Based again on a sea level rise of 1 m by the year 2100 (Section 8.2.1), and on the beach profiles measured in the Richards Bay study area, as well as the nearshore profiles determined from naval charts, the "Bruun rule" (1988) predicts recession of 43 m in this case (Table 9.3). (The depth of the profile of exchange, which is required in order to apply the equation, was determined to extend from +3 m to –15 m MSL, while the corresponding profile length was in the order of 770 m.) In the Richards Bay case, where large dunes are found, these may significantly reduce such horizontal shoreline erosion (assuming that they are maintained).

Table 9.3: Potential erosion according to Bruun's rule in the two study areas.

Study area	Potential erosion (m)
Southern Table Bay	**41**
Northern Table Bay	187
Richards Bay North	**43**

Step 5: Determine the coastal flooding elevations

- Determine extreme seawater levels, include SLR scenarios.

The tidal ranges for the South African coast including Table Bay and Richards Bay, are summarized in Table 5.1. Extreme South African seawater levels excluding tides (thus mainly due to wind and hydrostatic setup) have been analysed for all of the South African tidal stations, and the results (i.e. residuals for various return periods), also for Table Bay and Richards Bay, are summarized in Table 5.2. From the tidal level data in Table 5.1 and the extreme residual still-water level estimates in Table 5.2, the relevant data for the Cape Town and Richards Bay areas is summarised in Table 9.4.

Table 9.4: Summary of inshore seawater levels for the coastal study areas

Location	MHWS (m to MSL)	Residual setup 1:10 yr (m)
Cape Town	0.92	0.532
Richards Bay	1.10	0.534

- Model wave runup levels and determine coastal flooding elevations.

The extent of the Table Bay and Richards Bay setback line study areas have been pointed out before under Step 1. However, to demonstrate how the coastal flooding elevations can practically be determined for extended study areas, significantly larger areas were considered here, as described in the following paragraphs.

Runup modelling applying both the Mather and the Nielsen and Hanslow models, was conducted along a 26 km coastal area including the whole of the semi-shelterd Table Bay shoreline from Paarden Eiland adjacent to the Port of Cape Town, extending northwards up to the more exposed Koeberg area to the north of Cape Town (Figure 9.6; locations TBC1 to TBC58). Both runup models were used so as to compare and resolve or verify the results. As found in Section 5.3, the best results are obtained with the Nielsen and Hanslow (1991) wave runup model when wave heights are determined in the shallower nearshore area and then “reverse shoaled” to derive the equivalent deep-water wave heights as input. Thus the inshore wave heights along the 15 m isobath (as discussed in Step 4) were converted to “equivalent” offshore wave heights in this manner and then applied in the Nielsen and Hanslow model.

***Figure 9.6: The extended Table Bay study area - Paarden Eiland to Koeberg** (Google Earth image)*

In South Africa, spring tides (semi-diurnal) occur every two weeks, which means that the chances of storm waves coinciding with spring high tides are relatively high. Therefore, the input water levels were determined by combining spring high tidal levels (Table 9.4) with the 1-in-10 year residuals (wind and hydrostatic setups) from Table 9.4, with two SLR scenarios, namely the best estimate (mid-scenario) projections for 2050 and 2100 of 0.35 m and 1 m, respectively. Finally, these input water levels were combined with the modelled 1-in-10 year and 1-in-50 year wave runup heights, to yield two scenarios of extreme coastal flooding levels for each of the shoreline location points. The two scenarios of extreme coastal flooding levels, are as follows:

Scenario 1: The 1-in-10 year runup height + spring tide level (MHWS) + 1-in-10 year residual + 0.35 m SLR (2050 scenario) are combined. Note, this scenario was selected just to illustrate the result of a less extreme combination of all relevant components applicable to this coast. The intention is not that this scenario should be considered further in determining the coastal setback lines.

Scenario 2: The 1-in-50 year runup height + spring tide level (MHWS) + 1-in-10 year residual + 1.0 m SLR (2100 scenario) are combined, which is therefore an extreme combination of events. This is a suitable low risk planning scenario for high value or important coastal infrastructure with an expected long "service life". Informed also by research conducted in Australia (Cox, pers. com., 2012) this scenario is considered suitable for planning horizons in the order of 100 years (although the actual statistical return period is indeterminate). For strategic and critical infrastructure (e.g. nuclear power stations, major ports, hospitals, etc.), an even more extreme combination of events would be more appropriate. However, such specific strategic or critical cases should be addressed by means of detailed high-resolution, fine-scale investigations focussing on the circumstances of each case and location.

The results for the Table Bay area are shown in Table 9.5 (showing about 60 locations for this area). The last four columns give the extreme coastal flooding level outputs for the two scenarios as discussed above. The Table Bay shoreline location points cover an alongshore distance of about 26 km from the northern-most point (TBC1) adjacent to Koeberg, all along the Table Bay coastline up to the southern boundary (TBC52), which is just north of the Port of Cape Town (near Paarden Eiland) as indicated in Figure 9.6.

Table 9.5: Wave runup heights and coastal flooding levels for Table Bay - Koeberg shoreline

Location	Mather model		Nielsen & Hanslow model		Mather model SCENARIOS 1	Mather model SCENARIOS 2	Nielsen & Hanslow model SCENARIOS 1	Nielsen & Hanslow model SCENARIOS 2
	Wave run up above still water 1-in-10 yr	Wave run up above still water 1-in-50 yr	Wave run up above still water 1-in-10 yr	Wave run up above still water 1-in-50 yr	Total flooding level 1-in-10 yr + MHWS + residual + 0.35 m SLR (m MSL)	Total flooding level 1-in-50 yr + MHWS + residual + 1 m SLR (m MSL)	Total flooding level 1-in-10 yr + MHWS + residual + 0.35 m SLR (m MSL)	Total flooding level 1-in-50 yr + MHWS + residual + 1 m SLR (m MSL)
TBC1	3.6	4.1	3.6	3.8	**5.4**	**6.5**	5.4	6.3
TBC2	3.5	4.0	3.7	4.0	**5.3**	**6.4**	5.5	6.4
TBC3	3.5	4.0	3.7	4.0	**5.3**	**6.4**	5.5	6.4
TBC4	3.6	4.1	3.6	3.8	**5.4**	**6.6**	5.4	6.2
TBC5	3.6	4.1	3.6	3.8	**5.4**	**6.5**	5.4	6.3
TBC6	3.4	3.8	3.6	3.9	**5.2**	**6.3**	5.4	6.3
TBC7	3.1	3.5	3.4	3.6	**4.9**	**6.0**	5.2	6.1
TBC8	2.9	3.3	3.4	3.6	**4.7**	**5.7**	5.2	6.1
TBC9	2.9	3.2	3.4	3.7	**4.7**	**5.7**	5.2	6.1
TBC10	3.1	3.5	3.3	3.5	**4.9**	**5.9**	5.1	5.9
TBC11	3.3	3.7	3.4	3.5	**5.1**	**6.2**	5.2	6.0
MB Point								
TBC15	4.5	5.1	3.4	3.6	**6.3**	**7.6**	5.2	6.1
TBC16	4.1	4.6	3.4	3.6	**5.9**	**7.1**	5.2	6.1
TBC17	3.7	4.1	3.4	3.6	**5.5**	**6.6**	5.2	6.1
TBC18	3.4	3.8	3.3	3.5	**5.2**	**6.3**	5.1	6.0
TBC19	3.3	3.7	3.3	3.5	**5.1**	**6.1**	5.1	6.0
TBC20	3.2	3.6	3.0	3.2	**5.0**	**6.1**	4.8	5.6
TBC21	3.1	3.5	2.9	3.0	**4.9**	**6.0**	4.7	5.5
TBC22	3.2	3.6	2.9	3.0	**5.0**	**6.0**	4.7	5.5
TBC23	3.2	3.6	2.7	2.9	**5.0**	**6.0**	4.5	5.3
TBC24	2.0	2.3	2.6	2.8	**3.8**	**4.8**	4.4	5.2
TBC25	2.1	2.4	2.7	2.8	**3.9**	**4.8**	4.5	5.2
TBC26	2.4	2.7	2.8	2.9	**4.2**	**5.1**	4.6	5.4
TBC27	2.7	3.1	2.8	2.9	**4.5**	**5.5**	4.6	5.4
TBC28	2.8	3.2	2.7	2.8	**4.6**	**5.6**	4.5	5.3
TBC29	2.8	3.1	2.7	2.9	**4.6**	**5.6**	4.5	5.3
TBC30	3.2	3.6	2.7	2.9	**5.0**	**6.1**	4.5	5.3
TBC31	3.0	3.4	2.9	3.0	**4.8**	**5.8**	4.7	5.5
TBC32	3.0	3.4	2.9	3.1	**4.8**	**5.9**	4.7	5.5

Table 9.5 – continued:

Location	Mather model		Nielsen & Hanslow model		Mather model SCENARIOS 1	Mather model SCENARIOS 2	Nielsen & Hanslow model SCENARIOS 1	Nielsen & Hanslow model SCENARIOS 2
	Wave run up above still water 1-in-10 yr	Wave run up above still water 1-in-50 yr	Wave run up above still water 1-in-10 yr	Wave run up above still water 1-in-50 yr	Total flooding level 1-in-10 yr + MHWS + residual + 0.35 m SLR (m MSL)	Total flooding level 1-in-50 yr + MHWS + residual + 1 m SLR (m MSL)	Total flooding level 1-in-10 yr + MHWS + residual + 0.35 m SLR (m MSL)	Total flooding level 1-in-50 yr + MHWS + residual + 1 m SLR (m MSL)
Rehab								
TBC35	3.0	3.4	3.0	3.2	**4.8**	**5.9**	4.8	5.6
TBC36	2.8	3.2	2.8	3.0	**4.6**	**5.6**	4.6	5.4
TBC37	2.5	2.8	2.7	2.8	**4.3**	**5.3**	4.5	5.3
TBC38	2.7	3.0	2.5	2.6	**4.5**	**5.5**	4.3	5.0
TBC39	2.8	3.2	3.8	4.1	**4.6**	**5.6**	5.6	6.5
TBC40	3.2	3.6	3.5	3.7	**5.0**	**6.0**	5.3	6.2
TBC41	3.4	3.8	2.5	2.6	**5.2**	**6.3**	4.3	5.1
TBC42	3.3	3.7	2.5	2.6	**5.1**	**6.1**	4.3	5.1
TBC43	3.0	3.4	3.4	3.6	**4.8**	**5.8**	5.2	6.0
TBC44	2.9	3.3	2.6	2.7	**4.7**	**5.8**	4.4	5.2
TBC45	3.4	3.8	2.6	2.7	**5.2**	**6.3**	4.4	5.2
TBC46	3.7	4.2	3.5	3.7	**5.5**	**6.7**	5.3	6.2
TBC47	3.5	3.9	4.3	4.5	**5.3**	**6.4**	6.1	7.0
TBC48	3.8	4.3	2.8	3.0	**5.6**	**6.8**	4.6	5.4
TBC49	3.5	3.9	4.6	4.8	**5.3**	**6.4**	6.4	7.3
TBC50	3.3	3.7	2.4	2.5	**5.1**	**6.2**	4.2	5.0
TBC51	3.1	3.5	3.5	3.8	**4.9**	**5.9**	5.3	6.2
TBC52	2.9	3.3	2.9	3.0	**4.7**	**5.7**	4.7	5.5
TBC53	2.6	3.0	2.2	2.3	**4.4**	**5.4**	4.0	4.7
TBC54	2.5	2.8	2.2	2.3	**4.3**	**5.3**	4.0	4.7
TBC55	2.4	2.8	2.2	2.3	**4.2**	**5.2**	4.0	4.7
TBC56	2.3	2.6	2.2	2.2	**4.1**	**5.0**	4.0	4.7
TBC57	2.2	2.4	2.2	2.2	**4.0**	**4.9**	4.0	4.7
TBC58	2.0	2.3	2.2	2.2	**3.8**	**4.7**	4.0	4.7

The results from the wave runup models already include all the necessary sea-level and runup components, thus directly yield the coastal flooding elevations. A graphical representation of all the results for the extended Table Bay area is shown in Figure 9.7. The average difference between the flooding elevations determined by means of the two models is 8% (absolute value), which is relatively small. Best fit trend lines (RMS) for the predictions of both models are also indicated in Figure 9.7, further illustrating the good agreement between the two models, which further enhances confidence in the methods.

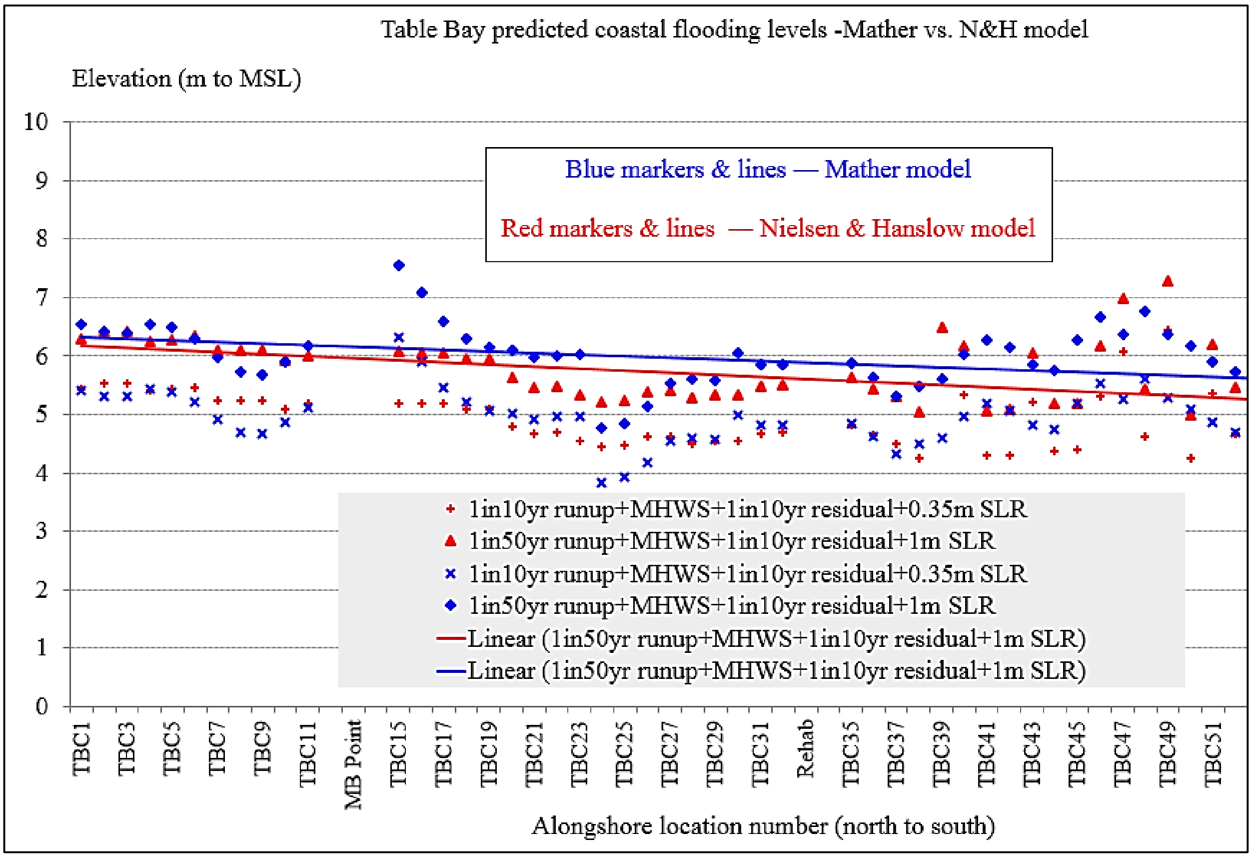


Figure 9.7: Coastal flooding predictions for Table Bay – Koeberg area

The same procedure and similar outputs were generated for each shoreline point within the extended Richards Bay coastal study area, which stretched from just north of the Port of Richards Bay to 24 km to the northeast (Figure 9.8: RBC184 to RBC234). Therefore, the same two scenarios of extreme coastal flooding levels were modelled, namely, by combining spring high tidal levels (Table 9.4) with the 1-in-10 year residuals for Richards Bay (Table 9.4), with two SLR scenarios (0.35 m and 1 m), and combining these with the modelled 1-in-10 year and 1-in-50 year wave runup heights.

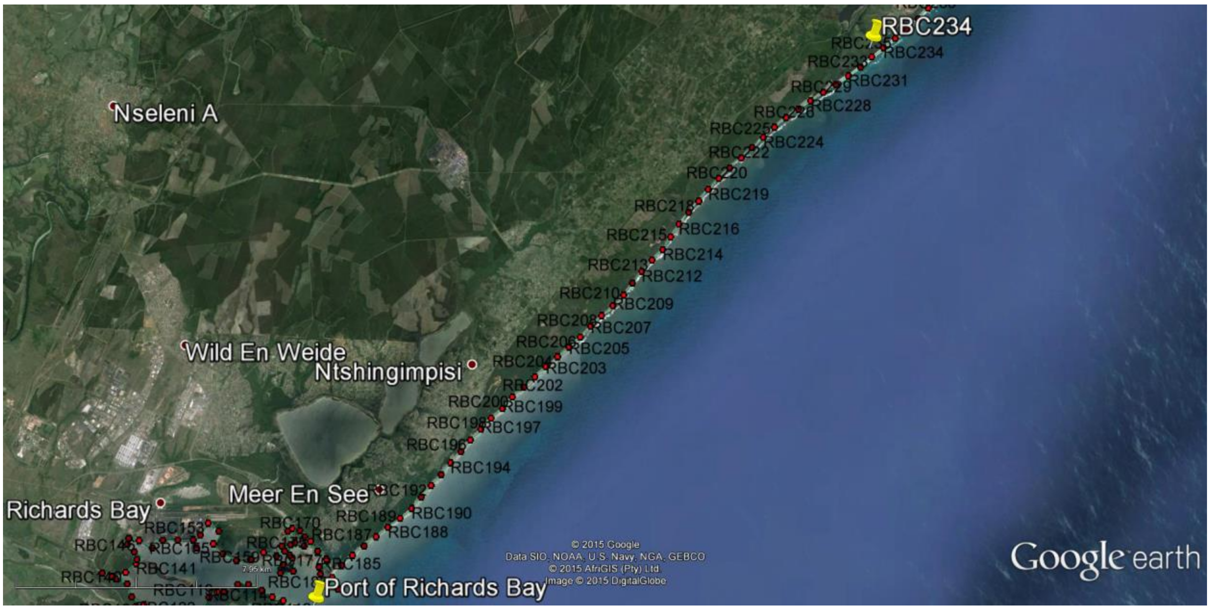


Figure 9.8: Aerial view of the extended Richards Bay coastal study area *(Google Earth image)*

The results for each of the 50 shoreline location points (at 500 m alongshore intervals) at Richards Bay are shown in Table 9.6. The last two columns give the extreme coastal flooding levels applying the Mather model, for the two scenarios as discussed above.

Table 9.6: Wave runup heights and coastal flooding levels for northern Richards Bay shoreline.

Location	Distance to -15 m contour (m)	Deep-sea wave height 1-in-10 yr (m)	Deep-sea wave height 1-in-50 yr (m)	Wave run up above still water Mather model 1-in-10 yr (m)	Wave run up above still water Mather model 1-in-50 yr (m)	Total flooding level 1-in-10 yr + MHWS + residual + 0.35m SLR (m MSL)	Total flooding level 1-in-50 yr + MHWS + residual + 1m SLR (m MSL)
RB Port						**Scenario 1**	**Scenario 2**
RBC184	1100	7.9	9.3	3.4	4.0	5.4	6.6
RBC185	1282	7.9	9.3	3.0	3.6	5.0	6.2
RBC186	1395	7.9	9.3	2.9	3.4	4.9	6.0
RBC187	1578	7.9	9.3	2.7	3.1	4.6	5.8
RBC188	1844	7.9	9.3	2.4	2.8	4.4	5.5
RBC189	2130	7.9	9.3	2.2	2.6	4.2	5.2
RBC190	1923	7.9	9.3	2.3	2.8	4.3	5.4
RBC191	1785	7.9	9.3	2.4	2.9	4.4	5.5
RBC192	1733	7.9	9.3	2.5	3.0	4.5	5.6
RBC193	1732	7.9	9.3	2.5	3.0	4.5	5.6
RBC194	1836	7.9	9.3	2.4	2.8	4.4	5.5
RBC195	1844	7.9	9.3	2.4	2.8	4.4	5.5
RBC196	1935	7.9	9.3	2.3	2.7	4.3	5.4
RBC197	2045	7.9	9.3	2.2	2.6	4.2	5.3
RBC198	2067	7.9	9.3	2.2	2.6	4.2	5.3
RBC199	2066	7.9	9.3	2.2	2.6	4.2	5.3
RBC200	2206	7.9	9.3	2.1	2.5	4.1	5.1
RBC201	1982	7.9	9.3	2.3	2.7	4.3	5.3
RBC202	1787	7.9	9.3	2.4	2.9	4.4	5.5
RBC203	1700	7.9	9.3	2.5	3.0	4.5	5.6
RBC204	1720	7.9	9.3	2.5	3.0	4.5	5.6
RBC205	1635	7.9	9.3	2.6	3.1	4.6	5.7
RBC206	1436	7.9	9.3	2.8	3.3	4.8	6.0
RBC207	1402	7.9	9.3	2.9	3.4	4.9	6.0
RBC208	1324	7.9	9.3	3.0	3.5	5.0	6.2
RBC209	1238	7.9	9.3	3.1	3.7	5.1	6.3
RBC210	1168	7.9	9.3	3.2	3.8	5.2	6.5
RBC211	1310	7.9	9.3	3.0	3.6	5.0	6.2
RBC212	1401	7.9	9.3	2.9	3.4	4.9	6.0
RBC213	1437	7.9	9.3	2.8	3.3	4.8	6.0
RBC214	1382	7.9	9.3	2.9	3.4	4.9	6.1
RBC215	1566	7.9	9.3	2.7	3.2	4.6	5.8

Table 9.6 – continued:

Location	Distance to -15 m contour (m)	Deep-sea wave height 1-in-10 yr (m)	Deep-sea wave height 1-in-50 yr (m)	Wave run up above still water Mather model 1-in-10 yr (m)	Wave run up above still water Mather model 1-in-50 yr (m)	Total flooding level 1-in-10 yr + MHWS + residual + 0.35m SLR (m MSL)	Total flooding level 1-in-50 yr + MHWS + residual + 1m SLR (m MSL)
						Scenario 1	Scenario 2
RBC216	1535	7.9	9.3	2.7	3.2	4.7	5.8
RBC217	1454	7.9	9.3	2.8	3.3	4.8	5.9
RBC218	1423	7.9	9.3	2.8	3.4	4.8	6.0
RBC219	1486	7.9	9.3	2.8	3.3	4.7	5.9
RBC220	1642	7.9	9.3	2.6	3.1	4.6	5.7
RBC221	1652	7.9	9.3	2.6	3.0	4.6	5.7
RBC222	1617	7.9	9.3	2.6	3.1	4.6	5.7
RBC223	1742	7.9	9.3	2.5	2.9	4.5	5.6
RBC224	1694	7.9	9.3	2.5	3.0	4.5	5.6
RBC225	1671	7.9	9.3	2.6	3.0	4.5	5.7
RBC226	1746	7.9	9.3	2.5	2.9	4.5	5.6
RBC227	1802	7.9	9.3	2.4	2.9	4.4	5.5
RBC228	1662	7.9	9.3	2.6	3.0	4.5	5.7
RBC229	1640	7.9	9.3	2.6	3.1	4.6	5.7
RBC230	1708	7.9	9.3	2.5	3.0	4.5	5.6
RBC231	1847	7.9	9.3	2.4	2.8	4.4	5.5
RBC232	1682	7.9	9.3	2.5	3.0	4.5	5.6
RBC233	1686	7.9	9.3	2.5	3.0	4.5	5.6
RBC234	1657	7.9	9.3	2.6	3.0	4.6	5.7

A graphical representation of all the results for the Richards Bay area is shown in Figure 9.9. Thus outputs were generated for each shoreline point within each of the two coastal areas modelled.

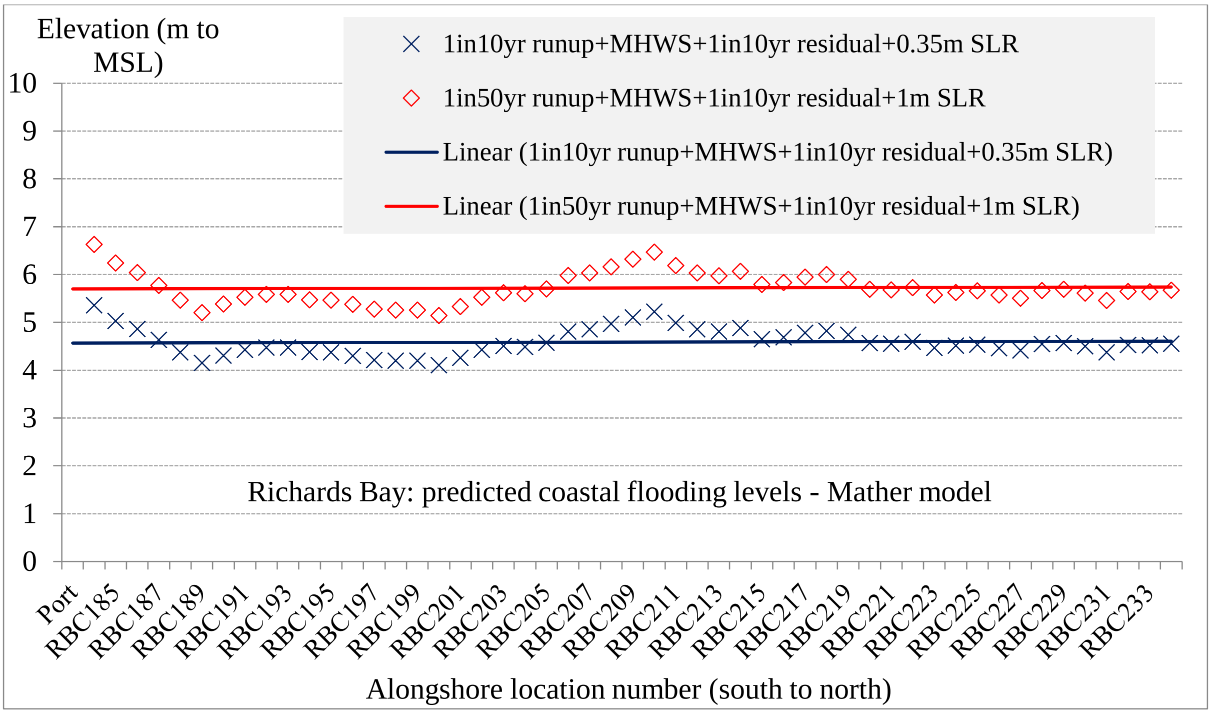


Figure 9.9: Coastal flooding predictions for extended Richards Bay area

Step 6: Determine the setback provisions for additional aspects

- Determine vegetation buffers for wind-blown sand.

The wind regimes at the two study areas are such that both study areas are seasonally subject to obliquely onshore winds over (at times) relatively wide dry beach areas. However, as discussed in Sections 8.3 and 9.3.3, the repective wind-blown sand transport potential and the characteristics of the respective coastal vegetation are of such nature, that the practical widths necessary to effectively trap wind-blown sediment are very different at the two study areas. Based on the recommendations provided in Section 9.3.3, a vegetation buffer of 15 m is applicable to the Richards Bay beaches, while a 35 m wide buffer is required for the Table Bay study area.

- Determine setback provisions for bluff/dune/cliff instability.

The dunes along the Table Bay study area are of relatively low height (virtually all much less than 10 m from base to crest), and have historically been observed not to be subject to dune slips.

However, the Richards Bay northern shoreline is backed by high dunes which are subject to large episodic dune slips. As discussed in Section 8.4, analyses of aerial photography reveal that dune slips in this area are associated with an average dune retreat of about 60 m, while a few very large slips resulted in dune retreat of about 100 m (with maximum alongshore extent of up to about 500 m). In this case, a minimum additional setback provision of about 110 m is warranted to safeguard against dune slip impacts. (Typical dune heights here are about 35 m to 40 m, which means the 110 m provision is in the order of three times the dune height, which is therefore also in line with the rule of thumb guideline provided in Section 9.3.3.)

Steps 1 to 6, as developed in Chapters 5 to 7, and compiled in Section 9.3 (Figure 9.2), have now been completed, demonstrating the application of the procedure and methods regarding all aspects of the erosion and coastal flooding levels components (the major focus areas of this thesis) at both study areas, including all the additional setback provisions for other physical coastal processes aspects. The pertinent results in terms of determining all aspects of setback distance provisions are summarised in Table 9.7, while the results regarding determination of all aspects of coastal flooding levels are provided in Tables 9.5 and 9.6 for the Table Bay and Richards Bay study areas respectively. The mapping of these results in the two study areas, is illustrated in Figures 9.10 and 9.11, which depict subsections of the two study areas.

Table 9.7: Summary of all setback distance provisions for 100 yr planning horizon.

Location	Long-term shoreline recession trend (m)	Predicted short-term erosion (m)	Additional storm erosion for wave height increase (Hs + 10% (m)	Sealevel rise erosion potential (m)	Vegetation buffer for wind-blown sand (m)	Provision for dune slip (m)	**Total setback distance (m)**
Table Bay	40	41	6	41	(35*)	0	**128**
Richards Bay	100	74	10	43	(15*)	110	**337**

* - regarding the vegetation buffers for wind-blown sand: in both these study areas it is practical to incorporate the whole required wind-blown sand buffer area within the area seaward of the erosion line, accepting that this buffer area will be partially washed away from time to time (as occurs naturally at these beaches). Limited erosion "damage" could sometimes be repaired through natural vegetation growth, but when more substantial storm damage has occurred, active human intervention will be required to rehabilitate the area.

Figure 9.10: Mapping of the 100 year "coastal processes" setback line for a portion of the Table Bay coast. *(Google Earth image)*

The 100 year "coastal processes" setback line mapped for the Table Bay coast (Figure 9.10) indicates significant potential impacts in this area. According to the predicted flooding levels and potential erosion (an example portion is mapped in Figure 9.10), sea storms combined with SLR effects will cause progressively worse problems for existing infrastructure and developments. As progressively higher sea levels are reached and the erosion events become more severe, the predicted runup increases and the potentially vulnerable areas become increasingly larger. Already, an extreme sea storm could cause major problems in the highly built-up areas near Blouberg. In addition, major transport infrastructure (the coastal trunk road) is also potentially at risk; all the more so as sea levels rise. (Note, however, that the runup methods applied assume a single slope, while the local topography in a few specific locations includes two alongshore dune ridges; having a dune crest followed by a trough followed by another crest, which will also affect, and possibly reduce the extent of the actual flooding.)

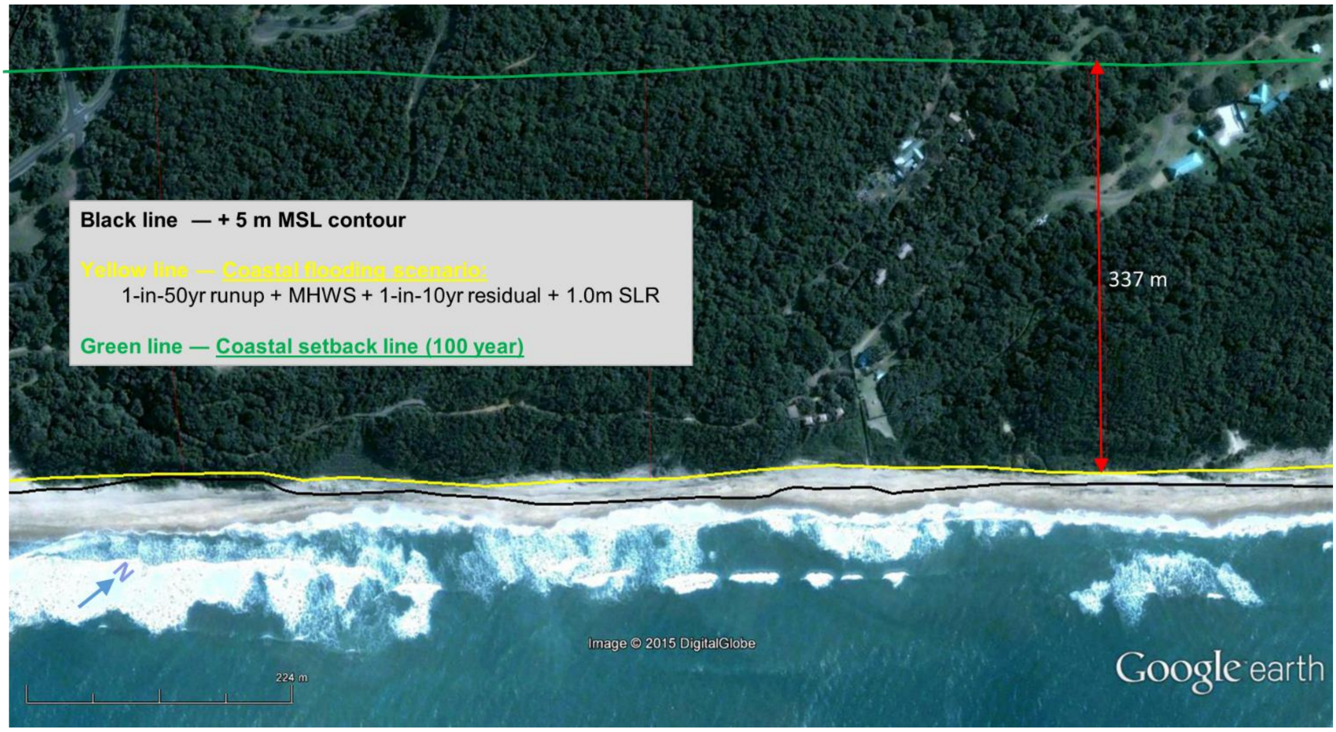


Figure 9.11: Mapping of the 100 year "coastal processes" setback line for a portion of the Richards Bay coast. *(Google Earth image)*

In the case of the Richards Bay area, the total "coastal processes" setback distance is very large (Figure 9.11). The implications for the northern Richards Bay coastline of the combined effects of ongoing shoreline recession, storm erosion events, progressive shoreline erosion due to SLR and high flooding levels, together with possible dune slips are extreme. This implies that the already high elevations reached by wave runup (and even higher future levels) could in addition in the long-term potentially shift landwards by up to about 340 m. It should however be noted that the provision for dune slip of 110 m and the 100 m provision for long-term recession trends, together result in a large additional setback distance of 210 m. The exceptionally large dunes (underlain by some more resistant mudstone and cohesive sediment layers) that are found here, may inhibit the rate of horizontal shoreline erosion. Never-the-less, the combined longer term impacts of higher storm wave runup levels and potential coastal erosion are anticipated to have major consequences.

Essential information for determining a setback line includes good topographical data of the shoreline and backshore areas (such as can be provided by means of, for example, a conventional topographical or LiDAR survey). The available topographical data for both the Table Bay and Richards Bay study areas was of relatively poor quality (2 m elevation contour intervals), which could result in some inaccuracies in mapping the + 5 m MSL contour lines and the predicted coastal flooding levels. The data was however adequate to clearly demonstrate the systematic practical application of the full setback line procedure and methods developed in this research.

Steps 7 to 10: Buffers or setback provisions for estuarine, ecological and social components, and merging of components

The foregoing applications and demonstrations of the procedure and methods to determine setback lines includes the major focus areas of this thesis (erosion and coastal flooding levels), as well as all the additional setback provisions for other physical coastal processes aspects (thus Steps 1 to 6). The remaining components of determining the complete coastal development setback line, entail the estuarine, ecological and social components (Steps 7, 8 and 9 respectively), while the final step (Step 10) entails combining all the different components. These latter components (Steps 7 to 10) are however not the focus of this thesis and their application will not be demonstrated for the two study areas considered here.

9.6 Conclusions

This chapter concerns putting all the setback line methodologies developed and aspects covered in the previous chapters together logically, and explaining how they should be practically applied to determine setback lines. Thus, the basic components of coastal development setback lines are catalogued and a compilation of the steps required to determine coastal development setback lines is provided. Based on those developed in the previous chapters, recommended procedures and methods for conducting/completing each of the steps required to determine coastal development setback lines are then summarized. The procedures and methods are aimed as far as possible at meeting the ideals/norms for "good and proper" and appropriate setback line determination in South Africa. While the methods have been developed specifically for application in South Africa, they are comprehensive and robust, and in a generic sense they could certainly be applied in other parts of the world too. The applicability of limited aspects, such as the Normal model (Section 6.2.2), would have to be proven in other parts of the world, while a few specific recommendations such as the wind-blown sand buffer width of 10 m to 40 m, would probably need to be adtaped in some areas.

For convenience, the major data requirements for determination of setback lines and some typical sources of such data are listed.

Case studies demonstrating practical application of all the recommended methods for the coastal processes components of determining coastal setback lines have been included in Chapters 4 to 8. These example applications were conducted at a variety of coastal locations representative of the different South African coastal areas, including, Durban Bight, Brighton Beach, Virginia Beach, Mossel Bay, Table Bay, Richards Bay, Saldanha Bay, and Walvis Peninsula. In addition, two specific case studies were conducted to demonstrate the systematic step-by-step application of the complete procedure and methods to determine "coastal processes" setback lines following the recommendations as provided in this chapter, namely one along the open coast at Richards Bay in northern Kwazulu-Natal on the Indian Ocean seaboard, and the other along the semi-shelterd Table Bay coast on the Atlantic Ocean seaboard along the southwestern African coast.

Practical guidelines for determining consistent and comprehensive/robust setback lines along the South African coast need to be drawn up as a matter of priority. It is believed that the recommended procedures and methods proposed in this chapter will contribute towards determining, setting and implementing appropriate, consistent/"standard" and robust practical methods of practice for conducting coastal setback line studies in South Africa.

Chapter 10: Summary and conclusions

10.1 Summary of findings

Literature review and discussion of methods

The South African coastline is rugged and exposed, with few natural bays, and consists of long stretches of sandy beaches interspersed by rocky sectors. For the present study the coastline has been sub-divided into five regions (Figure 2.9), on the basis of their morphological characteristics, general orientation and exposure to waves. The South African coast contains no muddy shorelines (other than inside some estuaries), nor barrier island coasts or delta coasts. Two types of sandy coasts occur most commonly, one being the generally high energy open shorelines often characterised by steeper slopes and more reflectives conditions consisting of medium to coarse sand. The other is characterised by milder slopes and more dissipative conditions often consisting of fine to medium sands, which is typically found in the more sheltered coastal embayments.

Implementation of the South African ICM Act (2008) has made it a legal requirement to determine coastal setback lines in all the South African coastal provinces. The purpose of a setback line is to:

- Protect private and public coastal property, including the natural environment;
- Demarcate safe areas, enable definition of areas at risk of being eroded or impacted by coastal processes, and enable the identification of infrastructure vulnerable to effects of sea level rise and inundation due to wave runup;
- Achieve conservation and sustainable development; and to
- Achieve other ICM considerations, e.g. biodiversity, coastal conservation, etc.

Thus, coastal development setback lines (or "coastal management lines") need to make provision for physical coastal/marine processes (especially erosion and coastal flooding), as well as "softer" more subjective issues and considerations, e.g. environmental, public access, heritage, sense of place, aesthetic, etc. This thesis describes the author's research concerning methods for the determination of coastal development setback lines in South Africa, and focuses strongly on the abiotic (geophysical) components of setback lines.

To date, the setback lines in South Africa have been determined in an ad hoc manner by practitioners on behalf of various municipal and provincial authorities in an inconsistent manner, with most of the South African coast still not yet covered. Both the literature review and recent setback line workshops held in South Africa have highlighted the lack of consistent methods to determine setback lines as well as the major confusion around how to proceed. In addition, the various technical methods that have been applied to determine setback lines in South Africa to date mostly have specific shortcomings. To alleviate these problems, appropriate setback line methods are sought for "data poor" environments, that can be efficiently applied in extensive study areas, but that are still sufficiently robust and defensible. In view of South Africa's generally very exposed, high energy, coastline (and the possibility of progressive climate change impacts), the escalating South African coastal development, and the above mentioned problems, the need for appropriate, practical and implementable methodologies to determine setback lines is clear.

Geophysical coastal hazards and spatial vulnerability

The most important drivers of risk to South African coastal assets from erosion and coastal flooding, are waves, tides and future sea level rise. A review is presented of coastal hazard assessment methods, from which a practical evaluation technique, developed for European coastal conditions but applicable to South African conditions and data availability, was identified. This technique was adapted and further developed (building on methods proposed by Theron *et al,* 2010a, 2012), to include additional forcing factors considered to be most relevant under South African conditions. In a case study, the coastal hazard assessment method was applied to Mossel Bay. The case study illustrates that the vulnerability assessment method developed, is very suitable for identifying hazardous coastal areas, and to quantify the relative vulnerability of each location along the shoreline. An output of this study is thus an adapted methodology suitable for assessing coastal hazards and mapping vulnerable coastal areas in South Africa. The findings from this research on geophysical coastal hazards are taken into account by ensuring that the recommended setback line methods do address all of the relevant coastal hazards (and their drivers).

Coastal flooding levels in South Africa

Significant drivers of high inshore seawater levels are tides, wind setup, inverse barometric setup, wave setup and, in future, sea-level rise, which all affect the still-water level at the shoreline. The additional significant component of extreme inshore seawater levels in the South African context is the wave runup. Based on an extensive literature review, it is concluded that the most appropriate (or 'central estimate') of sea-level rise (SLR) by 2100 is ~ 0.85 m to 1 m, with a plausible worst-case scenario of 2m and a low estimate of 0.5 m. The corresponding best estimate (mid-scenario) projections for 2030 and 2050 are about 0.15 m and 0.35 m, respectively.

Estimates are made of extreme values for realistic combinations of all the inshore seawater level components (described in Sections 5.2.1 to 5.2.4), as applicable to each South African coastal region. Based on these calculations and South African offshore wave conditions, estimates are made of the regional storm surge levels around the South African coast for the main offshore wave conditions. This provides a robust first-order coarse storm surge elevation assessment for the South African coastal regions, which feeds into coastal flooding determinations (Section 5.3.3), and also indicates the relative coastal flooding levels of the different South African coastal regions.

Following an extensive literature review and testing of 11 different wave runup models against local data, it is concluded that the three models of Nielsen and Hanslow (1991), Ruggiero *et al* (2001) and Mather *et al* (2011) are the best of the available models for application in South Africa. With an overall Root Mean Square Error of Prediction (RMSEP) of only 0.78 m, it is concluded that the model of Nielsen and Hanslow (1991) is generally the most suitable of the available models and is sufficiently robust with defendable results adequate for application in South Africa. Where only deep-water heights are known, or where data is lacking on the beach slope, the Mather *et al* model can be applied for sandy shorelines. An alternative value for coefficient C in the Mather *et al* model is suggested for application inside bays, to improve the applicability of the model to the full range of South African conditions. However, the Mather *et al* model is more applicable to exposed open coast locations (with steeper coastal slopes and more reflective conditions), such as are typical along the KZN coast, but also occur along portions of all five of the South African coastal regions (as discussed in Section 2.2.4). Overall, the Nielsen and Hanslow model is the most suitable for sandy shorelines; the best results will be obtained with significant wave heights determined at about 20 m depth or less and then "reverse shoaled" to give the equivalent deep-water wave heights as input. The adaptations to the Nielsen and Hanslow model (i.e. the new coefficient for beach slopes ≤ 0.06, and the "reverse shoaled" wave input from shallower depths) broadens the applicability of the model to both mild and steep sloped beaches (i.e. dissipative and reflective), thus catering for both the lower energy areas inside some South African bays (e.g. St Helena Bay, Table Bay, False Bay, Algoa Bay, etc.) and for the exposed high energy coasts (e.g. KZN, Cape south and west coasts, etc.). Thus, the Nielsen and Hanslow model can in this manner be applied to sandy shores within all five of the South African coastal regions (as characterised in Section 2.2.4). The Ruggiero model was also found to be a good alternative for the dissipative areas. The Nielsen and Hanslow wave runup model was applied in various case studies, yielding illustrative wave runup predictions for the South African coast. Further case studies applying the prediction methodology for extreme water levels, sea storms and wave runup, and illustrating climate change effects on wave runup, were conducted for Durban, Table Bay and Mossel Bay.

Shoreline changes and coastal erosion

Conventional numerical cross-shore sediment transport or morphology models are largely suited to detailed studies of small areas and face substantial practical difficulties with applications to large study areas (as discussed in Chapter 6). In view of the difficulties associated with applying conventional (2D or 3D) modelling and the accompanying need for extensive (and expensive) data collection and calibration, alternative approaches were pursued. Thus, new methods were developed and two alternative approaches are proposed, requiring less input data to quantify short-term shoreline erosion, and that are also suitable for larger scale applications.

It is concluded that the short-term shoreline variability of protected, moderately protected, and exposed, natural beaches in South Africa is mostly normally distributed. This can be used to predict the maximum landward movement over a selected period (say 50 years), based on the Normal model (Equation 6.1). The Normal model provided satisfactory results under a fairly wide range of conditions and it appears that the model is relatively robust. As topographic survey data is often not available, the utility of aerial photography data for predicting erosion was also investigated. In the test areas, the statistical analysis of the shoreline variability based on the aerial photography data, yielded good predictions of the short-term shoreline erosion that correspond well with the results based on more extensive and accurate topographic survey data, both using the Normal model.

The focus of the second approach was on the novel application of basic parameterizations, as the basis for developing a model capable of predicting the amount of erosion. The Sunamura and Horikawa (1974) formulation gave the best overall performance (of four basic parameterizations initially considered). The derived formulation (the “Parametric model”) for predicting cross-shore erosion distance is given in Equation 6.6. The predicted erosion distances based on the Parametric model were directly compared with the recorded erosion data. The general performance of the model appears to be acceptable. Additional model tests were performed and the satisfactory results attest to the veracity of the model. This led to the conclusion that the Parametric erosion model is relatively robust and provides satisfactory results under a relatively wide range of conditions.

Additional case studies were conducted to compare the Normal and Parametric erosion prediction methods and test their abilities by selecting two diverse new study areas, as well as by further comparing the results from these two models to those of a more complex modern erosion model (i.e. with a much stronger theoretical basis). The good results from the new study areas and good correspondence in both instances with the more complex modern model, in addition to the testing and validations described in Sections 6.2.2 and 6.2.3, are consirered to have sufficiently established the verification of the two new erosion models, attesting to their robustness and applicability for a variety of environments that are representative of the South African coast (and potentially even wider). Certainly the types of sandy coasts that occur most commonly is South Africa (as characterised in Sections 2.2 and 2.8) are well represented by these case studies and the various test sites used in Sections 6.2.2 and 6.2.3.

In determining setbacks it is also critical to identify any long-term shoreline erosion trends, yet this remains an aspect that is often neglected. An example is given (in Section 6.3) to illustrate why an assessment of long-term stability or possible erosional trends is so important.

A method to account for the additional protection provided by dunes in determining setback lines

The effects of dunes on determining setbacks are also investigated. It is found that the current methods of determining setback lines have not adequately taken dune effects into account. A widely applied cross-shore morphology model was utilized to compare and calibrate the effects of a variety of dunes on setback distances. A generalized dune volume versus setback line distance relationship was determined (Equation 7.1). It is suggested that the relationship in Equation 7.1 may be applicable for most of the South African coastline, as the test locations were specifically selected to be on different seabeds (on the Atlantic and Indian Oceans) and have different beach characteristics. Thus, the above relationship, which accounts for the impact of dunes on setback lines, is expected to be robust and generally applicable within South Africa. This research and the derived relationship can also be effectively employed to show (and quantify) easily the potential advantage of maintaining, rehabilitating or re-instating dunes as a natural shoreline protection measure against coastal erosion.

Other components and aspects of setback lines

Ideally a conservative planning horizon of 100 years should be considered for all coastal setback lines (although a planning horizon of 50 years would suffice for certain intensities or types of coastal development or infrastructure).

It was found that the Bruun rule is still the most suitable method to make a first estimate of possible erosion of ’soft’ sandy beaches as a consequence of sea level rise. In a case study, the combined effects were predicted of potential shoreline erosion with Bruun's rule, and higher wave runup from SLR with a 1-in-20 year sea storm on the Table Bay coast. Alternative methods to Bruun's rule for sandy shores are discussed, but none was found to be suitable for general practical application in South Africa. Methods of evaluating the flooding or erosion impacts of SLR along hard erosion-resistant shores and along "mixed sandy/rocky" shorelines are investigated, and estimation procedures are provided.

It has been determined that along the South African coast, the minimum practical width of vegetated buffer zones that is necessary to trap wind-blown sediment, effectively ranges from about 10 m to 40 m, depending on circumstances. The low end of this range (10 m to 20 m) is applicable to beaches found along the KwaZulu-Natal coast, while the top end of the rage (30 m to 40 m) is applicable to areas such as the Cape.

For dunes of more than 10 m in height it is advisable to involve geotechnical engineers and to conduct a slip circle analysis, especially if the slope is steeper than 1:6. Alternatively, a conservative setback from the dune crest of at least two to three times the dune height is required.

The general minimum additional erosion setback provision for hard erosion resistant "stable" shorelines in South Africa is here recommended as follows: Cliffs - 15 m landward from the edge of the cliff; High rocky shores - 6 m landward from the top (crest) of steep slopes; and Low rocky shores - 30 m landward from the natural coastal vegetation line. (In extreme situations this should be checked against coastal flooding simulations to ensure adequate setback provision has been made.)

Erosion setback requirements at estuary and river mouths are strongly informed by historic data and information on mouth changes and channel meandering. In South Africa, three basic options are currently employed for delineating "setback lines" or "buffer areas" along estuarine reaches inland of the mouth. The best guidance should be forthcoming in the near future from a report that is presently being drafted (Van Niekerk *et al*, 2014).

Coastal development setback lines (or "coastal management lines") also have to consider other important aspects, namely ecological and social components. Ecological components of coastal setback lines refer to ecological, biodiversity, and environmental conservation aspects, and related ICM requirements. Social components of coastal setback lines include consideration of and making setback provisions for: legal and zoning aspects, public access, aesthetic features, heritage, and shade.

Finally the coastal processes setback provisions are combined with the ecological, social and coastal zone management aspects or principles to derive the final coastal development setback line.

More discussions and specific recommendations and guidance for all these other components and aspects of setback lines are provided within the thesis.

Determining coastal development setback lines

All of the suitable methodologies and aspects required to determine setback lines are collated, and explanations are provided as to how they should be applied to determine setback lines. Thus, the basic components of coastal development setback lines are catalogued and a compilation of the steps required to determine coastal development setback lines is provided. Recommended procedures and methods for conducting or completing each of the steps required to determine coastal development setback lines are then summarized. The procedures and methods are aimed as far as possible at meeting the ideals or norms for "good and proper" and appropriate setback line determination in South Africa. For convenience, the major data requirements for determination of setback lines and some typical sources of such data are listed.

10.2 Conclusions

A European coastal hazard and vulnerability evaluation procedure (Coelho *et al*, 2006) provides a practical technique, applicable to South African conditions and poor data availability, and can be adapted and further developed (building on methods proposed by Theron *et al*, 2010a, 2012), to include additional forcing factors (e.g., wave exposure, sea level rise erosion potential, foredune height or volume) considered to be most relevant under local conditions. The adapted methodology developed in this research is suitable for assessing coastal hazards and mapping vulnerable coastal areas in South Africa. This ensures that all relevant coastal hazards are considered in determining setback lines.

It is concluded that the primary coastal processes components of setback lines concern coastal flooding elevations and coastal erosion (and therefore received primary attention in the research conducted for this thesis). Based on an extensive literature review and testing of 11 different wave runup models against local data, the three models of Nielsen and Hanslow (1991), Ruggiero *et al* (2001) and Mather *et al* (2011) were found to be the best of the available models for application in

South Africa. The Nielsen and Hanslow (1991) and Mather *et al* (2011) models are adequate for application in South Africa, but they should, however, be used with the adaptations recommended in this thesis to enhance their applicability to all of the South African coastal environments. The Ruggiero model was also found to be a good alternative for the dissipative areas, while the Mather model is less suitable in such environments. Conventional methods to predict short-term shoreline erosion face substantial difficulties in wide-scale applications, but new methods were developed in this research entailing two alternative approaches to predict erosion, which require less input data and are suitable for larger scale approaches. Both of the new erosion models, yielded good results for all of the case studies and the various test sites, which well represented the types of sandy coasts that occur most commonly in South Africa. Current methods of determining setback lines have not adequately taken dune effects into account, but a novel approach was developed here to quantify dune effects on normal shoreline erosion estimates.

Besides coastal flooding elevations and erosion, another eight aspects are important for determining coastal development setback lines, for which specific recommendations, suggestions and guidance are provided. Thus a comprehensive coastal development setback line methodology was drawn up including recommended procedures and methods for conducting or completing each of the required steps (and is detailed in Chapter 9).

The objectives of this research and thesis were achieved in that: remedies were developed for shortcomings in the methods that have been applied to date to determine setback lines in South Africa; appropriate setback line methods could indeed be developed for use in “data poor” environments that are efficient to apply in extensive study areas, and that are robust and defendable; recommendations are provided for appropriate, practical and implementable methodologies to determine coastal development setback lines in South Africa.

Overall, it can be said that new methods providing realistic results and a comprehensive procedure have been developed that are fit for the purpose of determining setback lines in South Africa.

10.3 Future Research

Through the research conducted for this thesis, various aspects were identified that are considered to be worthy of further research or that could contribute to refining the methods developed in this research. These are briefly discussed in the following paragraphs.

The Nielsen and Hanslow (1991) and Mather *et al* (2011) wave runup models seem to be less reliable for ‘flat’ (low-gradient) beach slopes (or dissipative conditions). Further validation with field data is required, or an alternative method could be sought in future research on this topic. Although the research conducted here favours the Nielsen and Hanslow model in most instances, it cannot be firmly concluded that this model is indeed preferred when the beach slope ($\tan \alpha$) is ≤ 0.06. Additional field data is required to test which of the two models would be preferred under such conditions. The general validity and applicability for South African conditions (and preferably even broader) of the Mather *et al* (2011) model should ideally be investigated further. This would also assist in providing better guidance on the selection of the value of the coefficient contained in the Mather formula from its current wide range. Prototype data should be collected on extreme inshore seawater levels, wave runup elevations and flooding elevations for testing and verification of prediction methods. Important input parameters in the investigation of these phenomena include coastal elevations and slopes. As LiDAR data becomes more readily available along the South African coast, this source could be employed to provide accurate data on coastal elevations and slopes over relatively large areas.

Currently there is no validated method to assess properly the joint probability of occurrence of tides, surge, runup, etc. along shorelines in South Africa. Further research relating to inshore seawater levels (in South Africa) is required to address the questions of: which metocean events and physical coastal processes are related, how are they related, and to what degree do these joint occurrences result in or affect extreme coastal flooding levels.

It was found that the Normal model (as proposed in this research) could be used to give reasonable predictions of extreme shoreline excursions resulting from erosional events. In future, alternative extreme distributions could also be considered, for example the (2 parameter) Weibull distribution (e.g. Carter *et al* 1986). Thus, it is suggested that further research be conducted to test whether other statistical distributions typically used for extreme value analyses in coastal engineering such as the Weibull, Exponential or the Method of Moments (Goda, 2000; Holthuijsen, 2007) might provide good predictions of extreme erosion.

Although the Parametric erosion model as developed in this research yielded good results, potentially more accurate results might be obtained with significant wave heights determined at about 20 m depth or less and then “reverse shoaled” to give the equivalent deep-water wave heights as input. Such adaptation of the inputs would allow for greater sensitivity of the model to local variations in wave conditions, which might yield more accurate predictions. This suggestion should be investigated, based on recorded data. Thus coastal profiles need to be monitored in various locations, ideally representative of all coastal types and characteristics found along South African shorelines. Although still relatively expensive, LiDAR surveys can provide accurate data on coastal profiles along extensive lengths of coastline and is becoming more available along the South African coast.

Climate change investigations have to date not provided sufficient clarity on plausible or ”realistic” scenarios of the future metocean climate off South Africa. Present climate change studies could be followed up by more focussed research in future aimed at addressing this need. In short, this could entail more focussed investigation of the potential effects of global warming on the regional wind and wave regimes around South Africa by utilization of regional climate change models (or possibly appropriately downscaled global climate change models) to assess predictions of changes in metocean climate regimes.

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Appendix 1 - Glossary

(Adapted from Voigt (1998), DEAD & P (2010), plus additions by the author, and as used in this thesis.)

Accretion	The accumulation of (beach) sediment, deposited by natural fluid flow processes or wind transport (aeolian transport).
Alongshore	Parallel to and near the shoreline; same as longshore.
Astronomical tide	The tidal levels and character that would result from gravitational effects, for example of the earth, sun and moon, without any atmospheric influences.
Bar	An offshore ridge or mound of sand, gravel or other unconsolidated material that is submerged (at least at high tide), especially at the mouth of a river or estuary, lying parallel to and a short distance from the beach.
Bathymetry	The measurement of depths of water in oceans, seas and lakes; also the information derived from such measurements.
Bay	A recess or inlet in the shore of a sea or lake between two capes or headlands, not as large as a gulf but larger than a cove.
Beach	A deposit of noncohesive material (e.g. sand or gravel) situated on the interface between dry land and the sea (or large expanse of water) and actively 'worked' by present-day hydrodynamic processes (i.e. waves, tides and currents) and sometimes by winds.
Beach erosion	The carrying away of beach materials by wave action, tidal currents, littoral currents or wind.
Beach profile	A cross-section taken perpendicular to a given beach contour, the profile may include the face of a dune or seawall and extend from landward of the dune, across the beach and seaward underwater to the about the outer edge of the surf zone.
Bed	The bottom of a watercourse or any body of water.
Benefits	The economic value of a scheme, usually measured in terms of the cost of damages avoided by the scheme or the valuation of perceived amenity or environmental improvements.
Buffer area	A parcel or strip of land that is designed and designated to remain in an undisturbed and natural condition to protect an adjacent aquatic, wetland

	or dune site from upland impacts, to provide habitat for wildlife and to afford limited access. Along the South African coast, there are many examples of where anthropogenic impacts have resulted in severe problems with wind-blown sand. An additional area is required to buffer the effect of wind-blown sand. In nature, these requirements are usually met by the presence of a vegetated foredune, or a wide, sandy area characterised by hummock dunes. Thus, a vegetated buffer zone/area with a certain minimum width is also required to trap wind-blown sand within the beach area. Buffer areas can also be employed for other coastal management functions, for example, conservation (/protection) of environmentally sensitive features or vegetation (i.e. biodiversity), or aesthetics or heritage issues. Such requirements are discussed more fully in Section 8.6.
Cay	A small, low island composed largely or coral or sand.
CD	Chart Datum
Cliff	A high, steep face of rock.
Climate change	“As a result of increasing atmospheric concentrations of carbon dioxide and other greenhouse gases from the burning of fossil fuels and other land use changes, the Earth’s climate is changing and is expected to continue to change over this century and beyond” (Rumble *et al*, 2005). Simply defined, climate change can refer to any long-term trend in mean sea level, wave height, wind speed, drift rate, and so forth.
Coast	A strip of land of indefinite length and width (may be tens of kilometres) that extends from the seashore inland to the first major change in terrain features.
Coastal management	The development of a strategic, long-term and sustainable land use policy, sometimes also called shoreline management.
Coastal processes	Collective term covering the action of natural forces on the shoreline and the nearshore seabed.
Coastal zone	The land-sea-air interface zone around continents and islands extending from the landward edge of the coast to the outer extent of the continental shelf. In its wider meaning it is often taken as extending landward up to where littoral processes are active or could have an effect (which could be some kilometres inland in certain areas).

Coastline	Commonly, the line that forms the boundary between land and water.
Conservation	The protection of an area or particular element within an area, accepting the dynamic nature of the environment and therefore allowing change.
Continental shelf	The zone bordering a continent, extending from the line of permanent immersion to the depth, usually about 100 m to 200 m, where there is a marked or rather steep descent toward the great depths.
Contour line	A line connecting points on a land surface or sea bottom that have equal elevation. It is called an 'isobath' when connecting points of equal depth below a datum.
Cross-shore	Perpendicular to the shoreline.
Cusps	Cusps are defined as a series of low mounds of beach material separated by crescent-shaped troughs spaced at more or less regular intervals along the beach face.
Debris line	A line near the limit of storm wave uprush marking the landward limit of debris deposits.
Deep water	In regard to waves, where depth is greater than one half the wave length. Deep-water conditions are said to exist when the waves are not significantly affected by conditions on the bottom.
Deep-water waves	A wave in water the depth of which is greater than one half the wave length.
Depth	Vertical distance from still-water level (or datum as specified) to the bottom.
Design storm	Coastal protection structures will often be designed to withstand wave attack by the extreme design storm. The severity of the storm (i.e. return period) is chosen in view of the acceptable level of risk of damage or failure. A design storm consists of a design wave condition, a design water level and a duration.
Design wave	In the design of harbours, harbour works, and so forth, the type/s of wave selected as having the characteristics against which protection is desired.
Direction of waves	Direction from which waves are coming.
Direction of wind	Direction from which wind is blowing.
Dunes	Above water accumulations of windblown sand on the backshore, usually in the form of small hills or ridges, stabilised by vegetation or

	control structures. (Underwater dunes area type of bed form indicating significant sediment transport over a sandy seabed.)
Duration	In forecasting waves, the length of time that the wind blows in essentially the same direction.
Ecosystem	Living organisms and the non-living environment interacting in a given area.
Erosion	Beach erosion is the carrying away of beach materials by wave action, tidal currents, littoral currents or wind (the latter is sometimes referred to as deflation).
Estuary	A semi-enclosed coastal body of water that has a free or intermittent connection with the open sea. The seawater is often measurably diluted with freshwater. The estuarine part of a river is affected by tides.
Event	An occurrence meeting specified conditions, for example damage, a threshold wave height or a threshold water level.
Fetch	The length of unobstructed open sea surface across which the wind can generate waves (generating area).
Fetch length	The horizontal distance (in the direction of the wind) over which the wind blows and generates seas or creates wind setup.
Gabion	Steel wire-mesh basket to hold stones or crushed rock to protect a bank or bottom from erosion.
Geology	The science that deals with the original history and structure of the earth as recorded in rocks, together with the forces and processes now operating to modify rocks.
Georeferencing	Establishing the location of an image in terms of map projections or coordinate systems.
High water	Maximum height reached by a rising tide. The height may be solely due to the periodic tidal forces, or it may have superimposed upon it the effects of prevailing meteorological conditions. Non-technically, also called the high tide.
High-water mark	In terms of the ICM Act (2008), it is defined as “the highest line reached by coastal waters”, which can be interpreted as being the wave runup (or approximately the debris line).
Littoral zone	This is a coastal zone where coastal processes have a direct influence, for example, longshore sediment transport and aeolian sediment

	transport.
Mean high-water springs (MHWS)	The average height of the high water occurring at the time of spring tides.
Mean sea level (MSL)	The average height of the surface of the sea for all stages of the tide over a 19-year period, usually determined from hourly height readings.
Ocean	The great body of salt water that occupies two thirds of the surface of the earth, or one of its major subdivisions.
Offshore	The zone beyond the nearshore zone where sediment motion induced by waves alone effectively ceases and where the influence of the seabed on wave action is small in comparison with the effect of wind.
Offshore wind	A wind blowing seaward from the land in the coastal area.
Outcrop	A surface exposure of bare rock not covered by soil or vegetation.
Overtopping	Water carried over the top of a coastal defence due to wave runup or surge action exceeding the crest height.
Peak period	The wave period determined by the inverse of the frequency at which the wave energy spectrum reaches its maximum.
Photogrammetry	The science of deducing the physical dimensions of objects from measurements on images (usually photographs) of the objects.
Port	A place where vessels may discharge or receive cargo.
Reach	An arm of the ocean extending into the land, usually a straight section of restricted waterway of considerable extent.
Recession	A continuing or net landward movement of the shoreline.
Refraction	The process by which the direction of a wave moving in shallow water at an angle to the bottom contours is changed. The part of the wave moving shoreward in shallower water travels more slowly than the portion in deeper water, causing the wave to turn or bend to become parallel to the contours.
Return period	Average period of time between occurrences of a given event.
Revetment	A facing of stone, concrete, and so forth to protect a scarp, embankment or shore structure against erosion by wave action or currents.
Rocks	An aggregate of one or more minerals rather large in size. The three classes of rocks are the following: (1) Igneous rock – crystalline rocks formed from molten material. Examples are granite and basalt. (2) Sedimentary rock – a rock resulting from the consolidation of loose

	sediment that has accumulated in layers. Examples are sandstone, shale and limestone. (3) Metamorphic rock – rock that has formed from pre-existing rock as a result of heat or pressure.
Runup	The rush of water up a structure or beach on the breaking of a wave. The amount of runup is the vertical height above still-water level that the rush of water reaches.
Sand	An unconsolidated (geologically) mixture of inorganic soil (that may include disintegrated shells and coral) consisting of small but easily distinguishable grains ranging in size from about 0.062 mm to 2 mm.
Scour protection	Protection against erosion of the seabed in front of the toe.
Sea defences	Works to prevent or alleviate flooding by the sea.
Sea level rise (SLR)	The long-term ascending trend in mean sea level.
Seawall	A structure built along a portion of a coast primarily to prevent erosion and other damage by wave action. It retains earth against its shoreward face.
Sediment transport	The main agencies by which sedimentary materials are moved are gravity (gravity transport); running water (rivers and streams); ice (glaciers); wind; and the sea through currents and wave action (for example, tidal currents and longshore drift). Running water and wind are the most widespread transporting agents.
Setback	Commonly used in CZM and coastal engineering terms as a required distance landward of a selected contour line (or the shoreline) to safeguard, for example, infrastructure from marine impacts (such as storm waves or erosion).
Shallow water	Water of such depth that surface waves are noticeably affected by bottom topography. Typically this implies a water depth equivalent to less than half the wave length.
Shoal	To become shallow gradually, proceeding from a greater to a lesser depth of water.
Shore	That strip of ground bordering any body of water that is alternatively exposed or covered by tides and/or waves. A shore of unconsolidated material is usually called a ‘beach’. ‘Shoreline’ is often used as the term for delineating between the land and the sea (e.g. selected as the 0 m to MSL contour line).

Significant wave height	Average height of the highest one third of the waves for a stated interval of time.
Significant wave period	Average period of the highest one third of the waves for a stated interval of time.
Soft defences	Usually refers to beaches (natural or designed) but may also relate to energy-absorbing beach control structures, including those constructed of rock, where these are used to control or redirect coastal processes rather than opposing or preventing them.
Spring tide	A tide that occurs at or near the time of new or full moon and that rises highest and falls lowest from the mean sea-level (MSL).
Still-water level	The time averaged elevation of the surface of the water. In deep water this level approximates the midpoint of the wave height. In shallow water it is nearer to the trough than the crest.
Storm surge	A concise definition of storm surge can be expressed as follows: "an abnormal rise of the mean seawater level generated by a storm and/or a meteorological event, over and above the astronomical spring high tides" (based on the NOAA (National Oceanographic Association of America) definition).
Surf zone	The zone of wave action extending from the water line (which varies with tide, surge, etc.) out to the most seaward of the zone (breaker zone) at which waves approaching the coastline commence breaking, typically in water depths of between 2 m and 15 m.
Survey, control	A survey that provides coordinates (horizontal or vertical) of point to which supplementary surveys are adjusted.
Survey, hydrographic	A survey that has as its principal purpose the determination of geometric and dynamic characteristics of bodies of water.
Survey, photogrammetric	A survey in which monuments are placed at points that have been determined photogrammetrically.
Survey, topographic	A survey that has for its major purpose the determination of the configuration (relief) of the surface of the land and the location of natural and artificial objects thereon.
Swash zone	The zone of wave action on the beach that moves as water levels vary, extending from the limit of rundown to the limit of runup.
Swell	Waves that have travelled a long distance from their generating area and

	have been sorted out by travel into long waves of the same approximate period.
Toe	The point of break in slope between a dune and a beach face.
Topographic map	A map on which elevations are shown by means of contour lines.
Updrift	The direction to which the predominant longshore movement of beach material approaches.
Vegetation line (coastal)	The coastal vegetation line is loosely defined here as an alongshore line marking the most seaward edge of substantial (i.e. ”woody” or ”semi-permanent”) natural coastal vegetation (i.e. usually not sparse or intermittent pioneer vegetation). Depending on in situ circumstances and on condition that the coastal vegetation has not been significantly affected by anthropogenic impacts, the vegetation line can be a good indicator of where the “natural” erosion line is located in the medium-term. However, a development setback line can usually not be located on the vegetation line, because this could in many instances lead to wind-blown sand problems (amongst others), or because this location may not be sufficiently conservative (i.e. unacceptably high risk in the longer term), or due to other requirements of CZM.
Wave crest	The highest part of the wave.
Wave direction	The direction from which the waves are coming.
Wave height	The vertical distance between the crest (the high point) and the trough (the low point) of the wave.
Wave hindcast	The calculation from historic synoptic weather charts of the wave characteristics that probably occurred at some past time.
Wave length	The distance in meters between equivalent points (crests or troughs) on waves.
Wave period	The time required for two successive wave crests to pass a fixed point.
Wave rose	Diagram showing the long-term distribution of wave height and direction.
Wave setup	Wave setup is defined as the time averaged super-elevation of the water surface over normal water elevation near the shoreline due to onshore mass transport of the water by wave action alone. In short, it is the elevation of the still-water level due to breaking waves.
Wave steepness	The ratio of wave height to length. (Not the slope between a wave crest

	and its adjacent trough.)
Wave train	A series of waves from the same direction.
Wave trough	The lowest part of the wave form between crests. Also that part of a wave below still-water level.
Wind rose	Diagram showing the long-term distribution of wind speed and direction.
Wind setup	The vertical rise in the still-water level (time averaged) on the down-wind side of a body of water caused by wind stresses on the surface of the water.
Wind waves	Waves formed and growing in height under the influence of wind.
World Geodetic System, 1984 (revised 2004)	An earth-fixed global reference frame used for defining coordinates when surveying and by GPS systems.

Appendix 2 – Wave runup field data sets

Table A: Wave runup field data from the Koeberg-Melkbos area (Bartels, 1985).

13-Sep-85			Wave height					
Runup elevation (m)	Tidal elevation (m)	Wave period (s)	Hs_{22} (m)	Hrms (m)	H_{022} (m)	H_{0deep} equivalent (m)	Peak wave period (s)	Beach face slope tan (α)
1.65	-0.23	9.16	1.89	1.36	1.80	1.96	10.71	0.091
1.29	-0.17	9.16	1.89	1.36	1.80	1.96	10.71	0.091
1.48	-0.09	9.16	1.89	1.36	1.80	1.96	10.71	0.091
1.86	0.00	9.16	1.89	1.36	1.80	1.96	10.71	0.091
1.59	0.07	8.76	1.96	1.36	1.87	2.04	10.24	0.091
1.87	0.14	8.76	1.96	1.36	1.87	2.04	10.24	0.091
1.66	0.22	8.76	1.96	1.36	1.87	2.04	10.24	0.091
2.37	0.31	8.76	1.96	1.36	1.87	2.04	10.24	0.091
2.32	0.38	8.76	1.96	1.36	1.87	2.04	10.24	0.091
2.40	0.44	8.76	1.96	1.36	1.87	2.04	10.24	0.091
2.15	0.47	8.70	1.81	1.28	1.72	1.88	10.17	0.091
2.45	0.51	8.70	1.81	1.28	1.72	1.88	10.17	0.091
2.59	0.55	8.70	1.81	1.28	1.72	1.88	10.17	0.091
2.16	0.59	8.70	1.81	1.28	1.72	1.88	10.17	0.091
2.56	0.62	8.70	1.81	1.28	1.72	1.88	10.17	0.091
2.19	0.66	8.70	1.81	1.28	1.72	1.88	10.17	0.091

20-Sep-85			Wave height					
Runup elevation (m)	Tidal elevation (m)	Wave period (s)	Hs_{22} (m)	Hrms (m)	H_{022} (m)	H_{0deep} (m)	Peak wave period (s)	Beach face slope tan (α)
1.15	0.04	8.51	1.82	1.30	1.74	1.87	11.65	0.040
1.21	0.00	8.51	1.82	1.30	1.74	1.87	11.65	0.040
1.12	-0.04	8.51	1.82	1.30	1.74	1.87	11.65	0.040
1.08	-0.07	8.51	1.82	1.30	1.74	1.87	11.65	0.040
1.05	0.10	8.57	1.73	1.24	1.66	1.78	11.73	0.040
0.92	-0.14	8.57	1.73	1.24	1.66	1.78	11.73	0.040
0.89	-0.17	8.57	1.73	1.24	1.66	1.78	11.73	0.040
0.82	-0.20	8.57	1.73	1.24	1.66	1.78	11.73	0.040
0.65	-0.22	8.57	1.73	1.24	1.66	1.78	11.73	0.040
0.68	-0.25	8.57	1.73	1.24	1.66	1.78	11.73	0.040
0.80	-0.26	8.63	1.96	1.38	1.88	2.01	11.81	0.040
0.90	-0.28	8.63	1.96	1.38	1.88	2.01	11.81	0.040
0.59	-0.30	8.63	1.96	1.38	1.88	2.01	11.81	0.040
0.67	-0.32	8.63	1.96	1.38	1.88	2.01	11.81	0.040
0.53	-0.33	8.63	1.96	1.38	1.88	2.01	11.81	0.040
0.78	-0.35	8.63	1.96	1.38	1.88	2.01	11.81	0.040
0.77	-0.36	8.63	1.96	1.38	1.88	2.01	11.81	0.040
0.80	-0.38	8.63	1.96	1.38	1.88	2.01	11.81	0.040
0.81	-0.40	8.63	1.96	1.38	1.88	2.01	11.81	0.040

24-Sep-85 Runup elevation (m)	Tidal elevation (m)	Wave period (s)	Wave height Hs_{22} (m)	Hrms (m)	H_{022} (m)	H_{0deep} (m)	Peak wave period (s)	Beach face slope tan (α)
2.32	0.41	9.45	3.27	2.44	3.12	3.34	11.99	0.091
3.30	0.46	9.45	3.27	2.44	3.12	3.34	11.99	0.091
3.11	0.52	9.45	3.27	2.44	3.12	3.34	11.99	0.091
2.84	0.44	9.45	3.27	2.44	3.12	3.34	11.99	0.091
3.10	0.70	8.96	3.81	2.67	3.64	3.93	11.37	0.091
2.81	0.84	8.96	3.81	2.67	3.64	3.93	11.37	0.091
3.17	0.78	9.45	3.81	2.75	3.64	3.90	11.99	0.091
2.94	0.78	9.45	3.81	2.75	3.64	3.90	11.99	0.091
3.27	0.88	9.45	3.81	2.75	3.64	3.90	11.99	0.091
3.66	0.90	9.02	3.59	2.53	3.43	3.70	11.45	0.091
3.14	1.06	9.02	3.59	2.53	3.43	3.70	11.45	0.091
3.50	0.94	9.02	3.59	2.53	3.43	3.70	11.45	0.091

26-Sep-85 Runup elevation (m)	Tidal elevation (m)	Wave period (s)	Wave height Hs_{22} (m)	Hrms (m)	H_{022} (m)	H_{0deep} (m)	Peak wave period (s)	Beach face slope tan (α)
0.20	-0.39	8.70	1.09	0.82	1.04	1.13	11.04	0.040
0.38	-0.38	8.70	1.09	0.82	1.04	1.13	11.04	0.040
0.46	-0.36	8.70	1.09	0.82	1.04	1.13	11.04	0.040
0.40	-0.24	8.70	1.09	0.82	1.04	1.13	11.04	0.040
0.36	-0.24	8.70	1.09	0.82	1.04	1.13	11.04	0.040
0.56	-0.14	9.45	1.06	0.76	1.01	1.08	11.99	0.040
0.52	-0.02	9.45	1.06	0.76	1.01	1.08	11.99	0.040
0.65	0.00	9.45	1.06	0.76	1.01	1.08	11.99	0.040
0.71	0.11	9.45	1.06	0.76	1.01	1.08	11.99	0.040
0.79	0.18	9.45	1.06	0.76	1.01	1.08	11.99	0.040
0.92	0.20	9.45	1.06	0.76	1.01	1.08	11.99	0.040
0.83	0.24	9.30	1.35	0.96	1.29	1.38	11.80	0.040
1.14	0.35	9.30	1.35	0.96	1.29	1.38	11.80	0.040
0.97	0.34	9.30	1.35	0.96	1.29	1.38	11.80	0.040
1.15	0.42	9.30	1.35	0.96	1.29	1.38	11.80	0.040
1.04	0.42	9.30	1.35	0.96	1.29	1.38	11.80	0.040
1.37	0.46	9.30	1.35	0.96	1.29	1.38	11.80	0.040
1.12	0.50	8.76	1.06	0.77	1.01	1.10	11.12	0.040
1.42	0.50	8.76	1.06	0.77	1.01	1.10	11.12	0.040
1.38	0.53	8.76	1.06	0.77	1.01	1.10	11.12	0.040
1.59	0.64	8.76	1.06	0.77	1.01	1.10	11.12	0.040
1.53	0.62	8.76	1.06	0.77	1.01	1.10	11.12	0.040
1.55	0.70	8.76	1.06	0.77	1.01	1.10	11.12	0.040
1.61	0.70	8.76	1.06	0.77	1.01	1.10	11.12	0.040
1.56	0.62	8.76	1.06	0.77	1.01	1.10	11.12	0.040
1.46	0.72	8.76	1.06	0.77	1.01	1.10	11.12	0.040
1.59	0.72	8.76	1.06	0.77	1.01	1.10	11.12	0.040
1.62	0.66	8.76	1.06	0.77	1.01	1.10	11.12	0.040

1.43	0.66	8.76	1.06	0.77	1.01	1.10	11.12	0.040
1.39	0.62	8.76	1.06	0.77	1.01	1.10	11.12	0.040
1.43	0.62	8.76	1.06	0.77	1.01	1.10	11.12	0.040
1.49	0.57	8.76	1.06	0.77	1.01	1.10	11.12	0.040
1.56	0.62	8.76	1.06	0.77	1.01	1.10	11.12	0.040
1.46	0.54	8.76	1.06	0.77	1.01	1.10	11.12	0.040
1.34	0.52	8.76	1.06	0.77	1.01	1.10	11.12	0.040

Table B: Wave runup field data from Table Bay Coast for storm of 31 August to 1 September 2008.

Significant wave height (H70s m)	Peak wave period (s)	Beach face slope tan (alfa)	Still water elevation (m)	Runup elevation (m)
10.29	16.60	0.065	0.95	2.80
10.29	16.60	0.107	0.95	4.88
10.29	16.60	0.043	0.95	4.03
10.29	16.60	0.100	0.95	5.18
10.29	16.60	0.125	0.95	4.59
10.29	16.60	0.056	0.95	3.62
10.29	16.60	0.022	0.95	2.99
10.29	16.60	0.049	0.95	4.04
10.29	16.60	0.100	0.95	5.04
10.29	16.60	0.063	0.95	4.53

(Cartoon courtesy of J Schoonees)